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DESIGN OF PLATE GIRDERS

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DESIGN
OF
PLATE GIRDERS

BY

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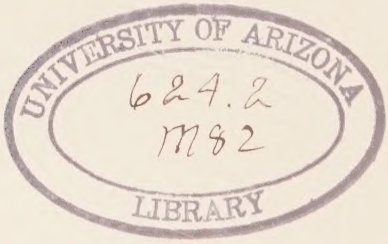

PREFACE

There are several books on bridge design in existence. All of them give more or less exact directions as to how to accomplish a specific purpose but do not discuss to any extent the general applicability of the rules laid down, nor do they as a rule sharply define the limitations of any given rule. This volume is the outcome of some six years of teaching the subject of Bridge Design at the Massachusetts Institute of Technology, and working at the same time for the Massachusetts Railroad Commission passing upon plans for new structures and inspecting existing ones. This is preceded by previous experience in bridge work, both with a bridge company and with a railroad. The author has found that students have, as a rule, and as is naturally to be expected, little or no idea of how to go about designing and less idea as to whether the product of their labors is a good, bad, or indifferent design. This book has been written with the idea of explaining clearly and in detail the reasons underlying designing, showing the assumptions made in given cases, and giving as far as possible alternative methods, indicating what seems to be the best way, and then allowing the student more or less of a choice as to the method to be pursued. The fundamental idea has been, not to lay down rules to be followed parrot-like, but to develop the ability of the student to see all the elements surrounding a given case and then to lead him to make an intelligent choice of a method to be pursued in his design.

Nearly all text-books and courses in structures lay a great amount of emphasis on the finding of stresses. There has been so much written on this subject that the author has purposely reduced the chapter on finding of stresses to what seems to him to be the irreducible minimum. Rivets have been treated somewhat more at length and in a different manner from that usually found in books on structural design. The theory of plate girder design has been set forth in detail and detailed designs of two different plate girders have been worked out with a careful discussion of each point as it arises. Railroad bridges are used in these designs because of the complications arising from rolling loads. The principles developed are of general application and can be extended to architectural work with no difficulty.

In order to develop the designs consistently, the specifications of the New York, New Haven & Hartford Railroad are used as a basis and then discussed in detail as the design is proceeded with.

The chapter on box girders has been worked out on similar lines, using a fixed load and without using any specifications. The material on deflections, contributed by Prof. W. H. Lawrence of the Massachusetts Institute of Technology, has never before been printed for general distribution.

The chapter by John C. Moses, engineer of construction of the Boston Bridge Works, has been incorporated to make the work complete, because no design is good unless it can be built at a reasonable cost, and a knowledge of shop possibilities is a necessity to every designer.

It is believed that the tables at the back of the book will be of material aid to the practicing engineer when designing plate girders both by the approximate and exact methods. The method of computing the tables of moments of inertia of angles and cover plates leaves the result subject to an error of 5 units in the last significant figure where there are four or more figures in a number. An error of this magnitude is of no consequence whatever in structural work and consequently it did not seem worth while to go to the large additional labor of eliminating such unimportant errors.

The author is indebted to many sources, but wishes especially to acknowledge his indebtedness to W. H. Lawrence, Professor of Architectural Engineering at the Massachusetts Institute of Technology, for permitting the use of his material on deflections and also for many helpful suggestions and for his encouragement; to Mr. W. H. Moore, Engineer of Bridges of the New York, New Haven & Hartford Railroad, for permission to reprint their specifications; to Mr. R. D. Bradbury, formerly an instructor in the Civil Engineering Department of the Massachusetts Institute of Technology, for many valuable suggestions; to Mr. Howard B. Luther, Instructor in Civil Engineering at the Massachusetts Institute of Technology, and to the author's father, L. T. Moore, formerly chief engineer of the Illinois Central Railroad, for reading the proof.

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DESIGN OF PLATE GIRDERS

CHAPTER I

STRESSES IN PLATE GIRDERS

The Evolution of the Modern Bridge.—The first bridge consisted in all probability of a log thrown across a stream. This type was probably afterward modified by using two logs, one at each side, with cross sticks upon them to form a floor. Where the distance to be covered was too great to use a single length of log, the natural step was to use two lengths of logs supported intermediately on piles or on a rough stone pier. This type of construction is evidently capable of indefinite expansion in both directions. It is called a pile or stringer bridge. Where, however, a stream is deep and rapid, the construction of so many pile bents or piers as are required becomes very uneconomical and some other means of supporting the floor and stringers becomes necessary. This is accomplished by supporting the ends of the stringers on cross-beams called floor-beams instead of on piers and then carrying the ends of these floor-beams on a truss, which may be defined for our purpose as an assemblage of parts joined together in the form of successive triangles. The first trusses were built of wood, but most trusses at the present time are built of steel. These trusses rest on abutments at each end, or in some cases where a crossing is very long and several successive truss bridges are required, there are intermediate piers each carrying the ends of two successive pairs of trusses. It is not within the scope of this work to go further into the questions involved in trusses, but to deal rather with plate girders which are the largest simple beams constructed. The design of such plate girders is treated in subsequent chapters and rests upon a few fundamental principles which render the designing very simple when they are once thoroughly grasped.

Finding of Loads.—The first thing to do in designing any beam is to find the loads it is intended to carry. Once these are known the determination of the bending moments and shearing

stresses at any given point or points becomes very simple. It is obvious that the girder must carry its own weight in addition to such loads as may come upon it and it is also plain that the weight of the girder cannot be known exactly until the design is completed. An estimate, or guess, at the weight must be made. This can ordinarily be done within fair limits of accuracy so that when the design is complete, little revision of the stresses and sections will be found necessary. More will be said about this later on. When the loads are moving, as in the case of a highway or railroad bridge, we must have some definite way of finding directly where to put the loads so that the maximum effect that they can produce may be found exactly and quickly.

Finding of Reactions: Statical Determination.—The next step after finding the loads is to find the reactions. In order to find the reactions we apply the three equations of equilibrium for coplanar non-concurrent forces. These equations are ΣH, ΣV, and $\Sigma M = O$.[1] They evidently enable us to find at once three unknown quantities. When we have two reactions, we have six unknown quantities to find. These are the magnitude, direction, and point of application of each of the two forces. We must then fix by some means or other three of these quantities. Generally we fix the points of application by assuming that the forces are applied at the geometrical centers of the areas of contact between the beam and its supports. If we then fix the direction of one of the supporting forces by some means we have disposed of three of the unknown quantities and can find the remaining three by applying the three equations of equilibrium given above. The direction of this supporting force is usually fixed by assuming that it is normal to the surface upon which the beam rests. Stated in another way we assume that the beam is resting at one end upon a frictionless surface. Sometimes, in long spans, one end of the beams is made to rest upon rollers which permit freedom of expansion and contraction and the other end may be supported upon a pin which serves to locate the point of application of the reaction quite definitely no matter how much the girder may deflect. The deflection of a girder under loads and supported on flat surfaces alters the point of application of the reactions by varying their distribution over the supporting surface. No attempt is ever made to compute this variation or

[1] See any work on mechanics.

to allow for it as its effect is to bring the reactions nearer to the loads and consequently to reduce the moments on the beam. In nearly all cases met with in practice, the loads and reactions are vertical which makes the application of the equation $\Sigma H = O$ fruitless and unnecessary. Either of the reactions is then found by taking moments of all forces on the beam about the other reaction The correctness of the arithmetical work involved may be checked by applying the equation $\Sigma V = O$ to all the forces and reactions. When a beam has only two supporting forces and they can be determined by the three static equations of equilibrium it is said to be statically determined. A beam which has more than two points of support has evidently more unknown quantities than there are equations of equilibrium and is said to be statically indeterminate. In such a case the supporting forces must be found by the aid of the "Theorem of Three Moments," or other equations involving the elastic properties of the beam. Such beams are rarely used in this country except in drawbridges. For methods of finding reactions in such cases the reader is referred to any of the standard works on structures. Most textbooks on mechanics of materials also give the "Theorem of Three Moments."

Method of Sections.—The method to be followed in finding the stresses at any point in a beam is to pass a plane through that point thereby separating the structure into two portions. One of these portions should then be removed and the equilibrium of the other one considered; applying such forces to the section cut by the plane as will hold the portion under consideration in equilibrium. These forces are always found by using the equations ΣH, ΣV, and $\Sigma M = O$, and in fact, the whole science of structures is built upon these three equations. The ability to apply them intelligently is all that is necessary to a thorough understanding of how to find stresses in statically determinate structures.

Moving Loads.—Where moving loads are concerned, the problem becomes somewhat more complicated, as it is necessary to know not only how to apply ΣH, ΣV, and ΣM to a given section but also where to locate the loads so that the greatest stresses that they can produce upon that section in moving over the structure will be found with certainty and ease. It is also necessary to locate the section in which the greatest of all the stresses on the bridge occur in order that it may be made of the proper size

to withstand them successfully. The usual moving load specified on an American railroad is one of Cooper's Series, so called from their originator. These loadings are known as *E*30, *E*40, *E*50, *E*60, etc., the *E* standing for engine and the numeral for one driving axle load in thousands of pounds. Some roads use an intermediate loading as *E*55, *E*56, etc. The "moment diagram" for the *E*50 loading is shown in Fig. 14, page 19. The only thing that needs explanation is the line entitled "moments in thousands of foot-pounds." These moments are the moments of all loads to the left of the load where the value of the moment is given about that load. The use of the diagram will be explained later.

For highway bridges and bridges carrying electric railways the following loads, taken from the Massachusetts Public Service Commission's Specifications, are suggested:

First. The weight of the structure itself.

In computing this, the weight of timber shall be taken as 4-1/2 lb. per foot board measure; and the weight of rails, steel guard rails, spikes, and bolts, shall be taken as not less than 100 lb. per linear foot of each track; but the total weight of the floor, above the stringers, shall not be assumed less than 300 lb. per running foot for each track.

Second. The live or moving load.

Stringer spans and the floor system of all trusses or girders shall be proportioned to carry a double-truck car weighing when loaded 50 tons with a total wheel-base of 25 ft. and a wheel-base for each truck of 5 ft. (See Fig. 43.)

Trusses and girders shall be proportioned to carry one car of the above type, or a uniformly distributed load, on each track. This uniform load shall be varied according to the length which has to be loaded by it to produce the maximum stress in the member in question. If this "loaded length" is 100 ft. or less, the load shall be 1500 lb. per linear foot of track; and if the "loaded length" is 300 ft. or over, the load shall be 1000 lb. per linear foot of track, and proportionally for intermediate lengths.

In highway bridges carrying electric roads the above specifications shall apply with reference to the loads upon the railway track. In addition, the following moving loads should be assumed upon the highway floor:

(*a*) For city bridges, subject to heavy loads:

For the floor and its supports, a uniform load of 100 lb. per

square foot of surface of the roadway and sidewalks, or a concentrated load of 20 tons on two axles 12 ft. apart, with 6 ft. between wheels. In computing the floor beams and supports, the railway load shall be assumed, together with either (1) this uniform load extending up to within 2 ft. of the rails, or (2) the above-described concentrated load alone.

For the trusses or girders, 100 lb. per square foot of floor surface for spans of 100 ft. or less, 80 lb. for spans of 200 ft. or over, and proportionally for intermediate spans. This uniform load is to be taken as covering the floor up to within 2 ft. of the rails.

(*b*) For suburban or town bridges, or heavy country highway bridges:

For the floor and its supports, a uniform load of 100 lb. per square foot, or a concentrated load of 12 tons on two axles 8 ft. apart; these loads to be used as described under (*a*).

For the trusses or girders, 80 lb. per square foot of floor surface for spans of 100 ft. or less, and 60 lb. for spans of 200 ft. or more, and proportionally for intermediate spans; to be used as described under (*a*). See (*d*.)

(*c*) For light country highway bridges:

For the floor and its supports, a uniform load of 80 lb. per square foot; this load to be used as described under (*a*). (See *d*.)

For the trusses or girders, 80 lb. per square foot of floor surface for spans of 75 ft. or less, and 50 lb. for spans of 200 ft. or more, and proportionally for intermediate spans; to be used as described under (*a*).

(*d*) All parts of the floor of a highway bridge should also be proportioned to carry a road roller weighing 15 tons, and having three wheels or rollers, the weight on the front roller being 6 tons, and the weight on each rear roller to be 4.5 tons. The width of the front roller is to be taken as 4 ft., and of each rear roller 20 in.; the distance apart of the two rear rollers to be 5 ft. center to center, and the distance between front and rear rollers 11 ft. center to center. In using this roller, the fiber stresses allowed shall be 30 per cent. above those specified in paragraph 18; and, if the stringers are not over 2-1/2 ft. apart on centers, each load shall be considered distributed equally on two stringers

(*e*) If ties or wooden floor beams are exposed to bending, the weight on one axle shall be considered as distributed equally upon three ties, if the latter are not over 8 in. apart in the clear. If they are farther apart, the load on each shall be found by assum-

ing an axle load to be distributed uniformly over a distance of four feet.

The total maximum stress in any piece shall be computed by adding together the dead and live stresses, the live loads being placed in the most unfavorable position, together with a percentage of the live stress to allow for impact and vibration. This added percentage shall be as follows:

	Per cent.
For floor beams and stringers,	25
For floor beam hangers,	40
For all counters,	40
For other members in trusses, and for main girders:	
When the "loaded length" is 20 ft. or less,	25
When the "loaded length" is 200 ft. or more,	10

and proportionally for intermediate lengths.

Lateral forces.

(*a*) A lateral force of 50 lb. per square foot on the unloaded structure, or of 30 lb. per square foot on the loaded structure, shall be provided for. The surface of the unloaded structure shall, in the case of a truss, be taken as twice the area of the vertical elevation of one truss, plus that of the floor; and in the case of a girder, as one and one-third times the vertical elevation of the structure. The surface of the loaded structure shall be that of the unloaded structure plus a vertical surface 10 ft. in height and 50 ft. long, the pressure on which is to be considered a moving load upon a car.

(*b*) In case a bridge is on a curve, a centrifugal force shall be assumed equal to 10 lb. per running foot for each degree of curve, acting at a height of 5 ft. above the base of the rail.

Longitudinal force.

A longitudinal force due to traction or the effect of brakes shall be provided for, equal to 20,000 lb. applied at the top of the rail.

Influence Line.—As we shall make use of the influence line in finding how to locate the loads upon a span so as to produce their maximum effect, we will proceed to define and illustrate how to construct it for simple cases. The influence line is a line showing the effect produced at a chosen place in a structure by a load of unity located at any or all points of a span. The effect produced may be a moment, shear, tension, compression, or any other function. It should be clearly understood that the effect produced is plotted not at the place or section under consideration,

but at the point where the load unity is located. The drawing of an influence line for a simple case will now be illustrated. Let it be required to draw the influence line for moment at the fixed point c of the beam ab, Fig. 1. The load unity is at any distance x from b. Other distances, etc., are as shown.

The influence line will be constructed on the line e–f as an axis. To find the moment at c we must pass a section through c and consider the equilibrium of one part of the beam after removing the other. In this case we will use the part ac, Fig. 1. Calling the left hand reaction R_1 the forces will be as shown in Fig. 2 so long as the load unity remains on the portion cb of the beam.

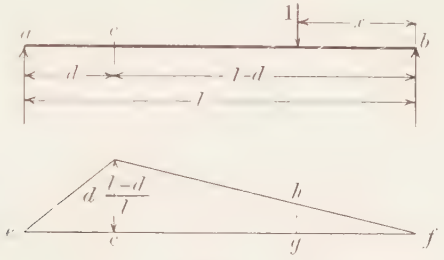


Fig. 1.

R_1 is found by taking moments about the end b of the beam and is $\frac{x}{l}$. The moment $R_1 d$ then becomes $\frac{dx}{l}$. The ordinates to the influence line then have the value $\frac{dx}{l}$ so long as the load 1 remains on cb and are plotted at the position of the load unity on the span or at a distance x from b. The equation $\frac{dx}{l}$ is evidently that of a straight line and when the load 1 is at c the ordinate of the line is $\frac{d(l-d)}{l}$. In a similar manner the line may be drawn for the portion between a and c and will be as shown in the figure. In drawing this part of the line, it will be simpler to consider the equilibrium of the part bc of the beam.

Now as to the use of the line. It is evident that a line may be drawn for a load of any magnitude P, and then the ordinate at any point g would equal the moment produced on the section c by the load P when P is at g. If the influence line be drawn for the load 1, however, as it usually is, the effect of a single concentrated load P at g, may be found by multiplying P by the

height gh measured to the scale to which the ordinates of the diagram are laid off. A little reflection will show that the influence line shows us where to place a single concentrated load so that it will produce the greatest possible effect. In the case in hand such a load should evidently be placed at the point c. It is also possible to ascertain the effect of a uniformly distributed load. To produce the greatest possible moment such a load should be distributed over the whole length of the beam, as the influence line lies entirely on one side of the reference axis ef. The moment caused by such a load is found by multiplying the area included between the line and the axis by the intensity of the uniformly distributed load. The intensity must be referred to the same units in which the span length l is expressed. (The student should find no difficulty in proving the above statement.) The following method of drawing an influence line for moment at any point in a simple beam is taken from Burr and Falk's "Influence Lines for Bridges and Roofs."

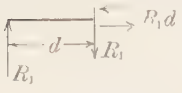


Fig. 2.

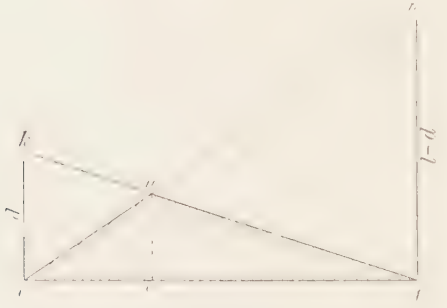


Fig. 3.

Let the problem be the same one that we had before. The method of procedure is as follows:

Erect at e, Fig. 3, a line ek, equal to $ec = ac$, Fig. 1, in length and at f, a line fm equal to $cf = cb$ in length. Join the upper ends of these lines to the opposite ends of ef. The shaded portion is the influence line. To show that it is exactly like the one we drew before, it is only necessary to show that the height cn equals $\frac{d}{l}(l-d)$ in both the triangles efm and efk.

In the triangle efm

$$\frac{cn}{fm}=\frac{ec}{ef} \text{ or } \frac{cn}{l-d}=\frac{d}{l} \text{ or } cn=\frac{d}{l}(l-d)$$

In the triangle efk

$$\frac{cn}{ek}=\frac{fc}{ef} \text{ or } \frac{cn}{d}=\frac{l-d}{l} \text{ or } cn=\frac{d}{l}(l-d)$$

Influence lines for shear are equally simple in their derivation for a simple beam. A simple method of drawing them will be given. It should first be explained that in finding shears at a section a concentrated load is never considered to be located exactly at the section, but is always placed an infinitesimal distance to one side or the other of the section. If the load were taken exactly at the section, the plane used in making the sec-

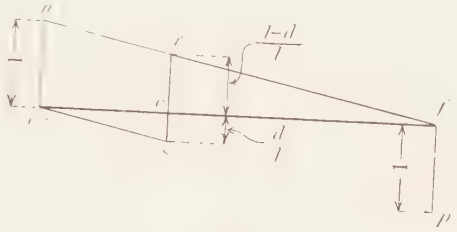


FIG. 4.

tion would pass through the load and there would be some uncertainty as to what portion of the load should be considered to be on either side of the section. This infinitesimal distance is a conception for convenience and is, of course, not computed in figuring stresses. The influence line for reaction at the left-hand end of a beam is evidently the same as that for shear an infinitesimal distance from the reaction, and is found to be *enf* in Fig. 4. It should be explained that the shear at any section is found by passing a plane through the section and considering the equilibrium of one portion after removing the other. It is called positive when the resultant of all the forces to the left of the section acts upward or when the resultant of all the forces to the right of the section acts downward. It is called negative when the resultant of all the forces to the left of the section acts downward or when the resultant of all the forces to the right of

the section acts upward. These distinctions are consistent, but entirely arbitrary. The student should be sure that he fully masters them before proceeding. Now let us draw on Fig. 4 the influence line for shear on a section an infinitesimal distance to the left of the right abutment. It will evidently be *epf* and *fp* will be negative and will equal 1 and hence should be plotted below the line. Now at *c* draw a perpendicular to *ef* cutting *nf* at *r* and *ep* at *s*. Then *frcsef* is the influence line for shear on the section *c*. To prove this, consider the equilibrium of the portion *ac* of the beam as before (see Fig. 1). So long as the load is on *cb*, the shear on the section is equal to the left-hand reaction R_1 and equals $\frac{x}{l}$ becoming $\frac{l-d}{l}$ when the load 1 is an infinitesimal distance to the right of the section. Similarly when the load comes on from the left on the portion *ac*, the height of the line is equal to $\frac{x}{l}$ where x is measured from the left-hand end and becomes $\frac{d}{l}$ when the load is an infinitesimal distance to the left of the section. The points *r* and *s* evidently lie on *fn* and *ep* respectively, hence our construction is correct.

Effect of Floor-beams.—Bridges, even those of the plate girder type, are in very many cases constructed with stringers and floor-beams. This construction alters somewhat the shape of the influence lines and must be thoroughly understood. In such cases all the live loads and a considerable proportion of the dead loads are carried by the stringers and distributed by them to the floor-beams. The floor-beams then carry the load to the girders. Each stringer is to be considered as a simple beam whose reactions are carried by the adjacent floor-beams. The floor-beams in turn carry such loads as are brought to them by the stringers together with their own weight; and their reactions are carried by the girders or trusses. Save for their own weight, the girders carry only such loads as are brought to them by the floor-beams. The points where the floor-beams are joined to the girders are called panel points. The distance between adjacent floor-beams is called a panel. A load upon a stringer is then distributed between two adjacent floor-beams and by the floor-beams is divided between the girders.

Figure 5 shows a plan of a typical "open-floor" through plate girder railroad bridge. The ties which support the rails

rest upon the tops of the stringers. They are omitted from the figure to avoid confusion. Suppose a load of unity were applied to each of the points "a" on two of the stringers 3 ft. from their

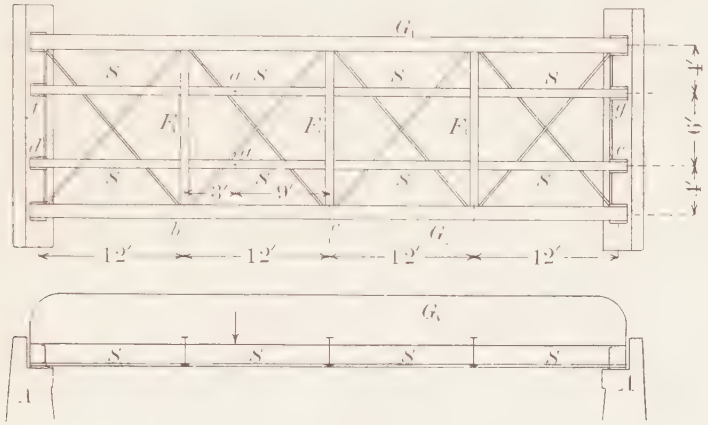


Fig. 5.

ends. The reactions on each of the stringers would be as shown in Fig. 6. The loads and reactions on floor-beams F_1 and F_2 would be as shown in Fig. 6. The loads and reactions on girders G_1 and G_2 would be as follows:

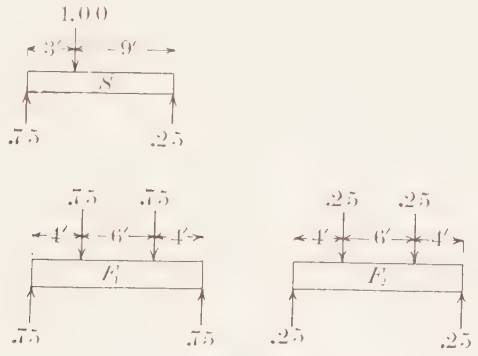


Fig. 6.

These figures are shown to illustrate the manner in which a load is carried and distributed. It should be noted that the reactions at the ends of the girders (Fig. 7) are the same as though the loads 1 at a were carried directly to the girders by a cross-beam

instead of passing through the intermediary stringers and the floor-beams F_1 and F_2. A little reflection will show that, if its own weight be neglected, the girder can only receive loads where the floor-beams are fastened to it and consequently with any given arrangement and position of loads the shear at all points on a

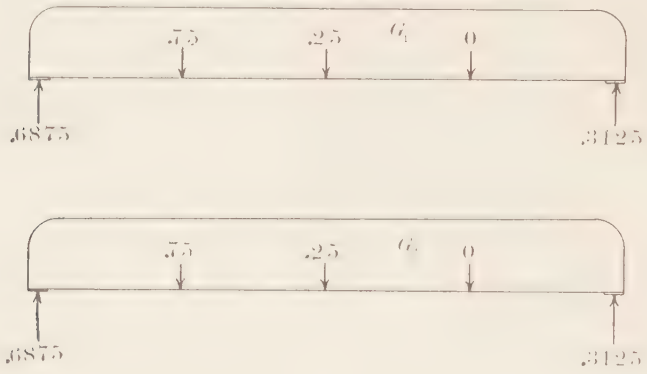


Fig. 7.

girder between adjacent panel points will be a constant quantity. Therefore, we may speak of finding the shear in a panel of a girder, or of drawing the influence line for shear in a panel of a girder, and it does not matter which point in a panel we consider. The moment influence line could not be drawn in the same way, however, as it would be different at different points within the panel. This can readily be seen by considering that the distance from the section under consideration to the reaction enters into the determination of the moment. As the loads are applied to the girder only at the floor-beam connections it is usually unnecessary to draw influence lines for moment at any other than these points because the maximum moment always occurs under a concentrated load.

Fig. 8.

The influence line for shear in the panel *bc* of the girder G_2 (Fig. 5) for a load of 1 on each of the lines of stringers *de* and *fg* is as shown in Fig. 8. It is most easily drawn by considering that the left-hand reaction would change from 0 to 1 for that part of the load 1 which comes to the girder 2 as the load moves across the

span. Similarly for the right-hand reactions. Connect the points d and e by a straight line. The points d and e are directly below and above the panel points b and c in the girder. The load unity in Fig. 5 is placed 9 ft. from the center of the bridge. It will be noticed that the influence line crosses the axis at a point o in the panel which in this case is 8 ft. from the point c. The point o is called the "neutral point." If a load be located at this point on a stringer it will cause no shear on the girder in the panel. If located to the right of it, it will cause a positive shear; if to the left it will cause a negative shear in the panel.

There is a neutral point in every panel except the end ones. Its location is different in every panel, but is easily found by a construction similar to Fig. 8 and by computing the distance co from the similar triangles oec and obd.

In drawing influence lines for bridges of this type, it is customary to assume, as we have done, that there is a load of unity on *each* line of stringers and that these loads are always kept abreast. This assumption can properly be made only when the bridge is symmetrical in plan about a longitudinal axis. When it is not symmetrical, as for instance in the case of a skew bridge, it may be necessary to draw the influence line for a load of unity on each line of stringers. The student should understand that stresses are not found by the direct use of influence lines in ordinary cases. They should, however, be thoroughly understood as their use is of material aid in skew and irregular structures.

Maximum Moments.—It is evidently a very simple matter to find the maximum moments on a simple beam when a uniformly distributed load is used, as it is merely necessary to load the whole beam. The proper location of a series of concentrated loads does not appear so simple at first sight although we shall see that it does not involve any serious difficulties. Suppose it is required to find the maximum moment which a given series of movable loads can produce at a given point. Draw the influence line for moment at the point and let the loads be as shown in Fig. 9.

We know that the maximum moment at c will occur when a load is at c but the question is which load should be at c? Let R_1 (not shown) be the resultant of all loads on the span to the left of c and R_2 (not shown) be the resultant of all loads on the span to the right of c. Then as the loads are moved from right to left a distance y the moment will increase provided the increase

in moment $R_2 y \frac{d}{l}\left(\frac{l-d}{l-d}\right)$ is greater than the decrease in moment

$$R_1 y \frac{d}{l}\frac{l-d}{d}.$$

Now
$$R_2 y \frac{d}{l}\frac{l-d}{l-d} > R_1 y \frac{d}{l}\frac{l-d}{d}$$

can be reduced without altering the inequality as follows:

$$\frac{R_2}{l-d} > \frac{R_1}{d}$$

Interpreting this, it means that if $\frac{R_2}{l-d} > \frac{R_1}{d}$ the moment will be increased by moving up the loads toward the left. If $\frac{R_2}{l-d} = \frac{R_1}{d}$, the moment will be unchanged and if $\frac{R_2}{l-d} < \frac{R_1}{d}$ the moment will be decreased. In the last contingency to produce the maximum

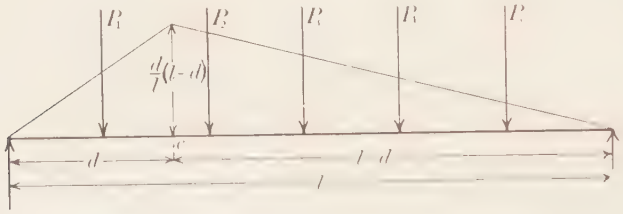


Fig. 9.

effect we should move the loads toward the right. Note that $\frac{R_2}{l-d}$ is the *average load* on the right of the section and that $\frac{R_1}{d}$ is the *average load* on the left of the section. We have then the following rule: Locate the loads on the span so that a heavy load is near the section and other loads are to the right and left of it. Place the heavy load just to the right of the section and find the average loads to right and left. If the average load on the right is found to be greater than that on the left, move the loads so as to place the heavy load just to the left of the section and again find the average loads to right and left. If the average on the left is now greater than that on the right, a maximum moment will be found with the load at the section. If on the contrary the average right is still greater than the average left,

move up the next load. It will evidently be unnecessary to try this load to the right of the section; simply move it at once just to the left and find the average loads. Continue this process until a load is reached at which the average changes from greater on the right to greater on the left as the load crosses the section.

Absolute Maximum Moment.—It is also often necessary to determine on a simple beam the point at which the greatest possible maximum moment of all maxima on the beam will occur. This is called the absolute maximum moment. It is readily found as follows. Let a series of loads be upon a beam as shown in Fig. 10 and let their resultant be R. It should be borne in

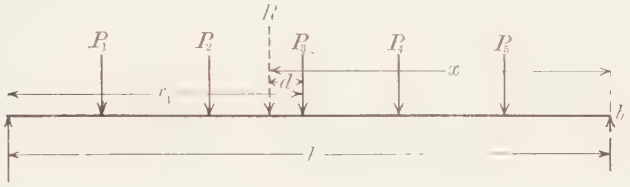


FIG. 10.

mind that the maximum moment will occur under a load, but the question is under which load? Suppose we try the load P_3. For any position of loads, R will be some distance x from b and the moment under P_3 will equal $\frac{Rxx_1}{l} - M$ where M is the moment of P_1 and P_2 about P_3. M is evidently a constant quantity. R is evidently a fixed distance d from P_3 so long as no loads go off or come on the span which contingency would have the effect of altering R in position and magnitude. We must investigate the expression $\frac{Rxx_1}{l} - M$ for a maximum. To do this we must express x_1 in terms of x

$$x_1 = l + d - x$$

substituting we have $\frac{Rx(l+d-x)}{l} - M$

letting this equation equal y and differentiating with respect to x we have

$$\frac{dy}{dx} = R(l+d-2x)$$

putting $\frac{dy}{dx}=0$ we have $R(l+d-2x)=0$

or $l+d-x=x$

or $x_1=x$

That is, if the loads are so located that the center of the span is halfway between the resultant and the load under consideration, the absolute maximum moment for that load will be found under it. All the loads on the span might be tried in this way and an absolute maximum found for each load. This is evidently unnecessary. All that is ever necessary is to find the absolute maximum moment under each of the loads adjacent to the resultant. Of these two loads the one nearer to the resultant gives the maximum in nearly every case.

Position of Loads for Producing Maximum Shear in a Panel. The rule which will be developed for finding the position of a series

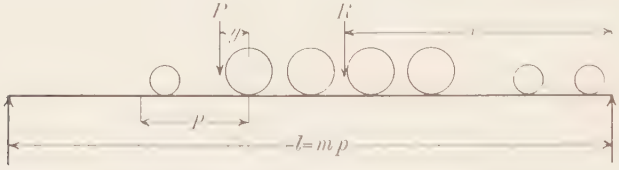


FIG. 11.

of concentrated loads which will produce maximum shear in a panel will be applicable to bridges having either equal or varying panel lengths and locomotive wheel loads will be used. (See Fig. 11.)

Let p = length of panel in which maximum shear is desired.

$m=\frac{l}{p}$: where panel lengths are all equal m = number of panels in bridge.

l = span $=mp$.

R = resultant of all loads on span.

P = resultant of loads in panel under consideration.

Q = resultant of loads to left of panel, if any.

V = shear in panel.

Then with the loads in the position shown

$$V=\frac{Rx}{l}-\frac{Py}{p}\quad Q$$

If the loads be moved to the left a distance dx the shear will change by an amount

$$dV = \frac{R dx}{l} - \frac{P dx}{p}$$

As we wish to find the maximum value of V, we may equate its first derivative dV to 0

$$dV = \frac{R dx}{l} - \frac{P dx}{p} = 0$$

or

$$\left(\frac{R}{l} - \frac{P}{p}\right) dx = 0$$

for this equation to be true $\frac{R}{l} - \frac{P}{p}$ must equal 0

or

$$\frac{R}{l} = \frac{P}{p} \text{ or } R = P\frac{l}{p} = Pm.$$

To state this rule in words:

For maximum shear in a panel locate a load at the end of the panel and multiply the sum of all the loads in the panel, including the one at the panel point, by the number of panels in the span. If the product be equal to or greater than the total load on the span a maximum shear will be produced by this load at this section. If the product be less than the total load on the

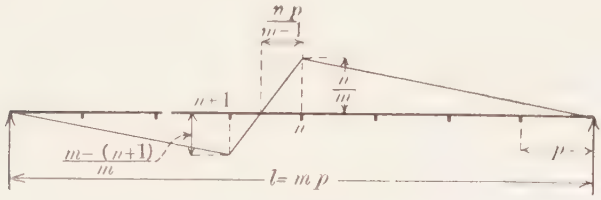


FIG. 12.

span move up the next load to the panel point and go through with the process again.

It is evident that as much of the load which is exactly at the panel point may be counted as will make

$Pm = R$, and it is not necessary to find what portion of that load will exactly satisfy the equation, as the first load which we come to in moving up the loads which will make $Pm \geq R$ will give the proper position for maximum shear in the panel.

This method is equally applicable to uniform loads.

Let us consider the applicability of the rule to the case of a

bridge of m equals panels under a uniform load as shown in Fig. 12. The influence line is drawn for the $(n+1)^{th}$ panel from the right end. The uniform load should evidently extend up to the neutral point in the panel.

Distance from n to neutral point of panel is

$$\frac{n}{m}+\frac{\frac{m}{m-(n+1)}}{m}\,p=\frac{np}{n+m-n-1}=\frac{np}{m-1}$$

If w be the intensity of the uniform load per foot of span the load in the panel will equal $\frac{wnp}{m-1}$ and the total load on the span will equal $wnp+\frac{wnp}{m-1}$. For our rule to hold true the following must be a true equation

$$\frac{mwnp}{m-1}=wnp+\frac{wnp}{m-1}$$

Multiplying both sides by $m-1$ we have

$mwnp=mwnp-wnp+wnp$ which is evidently an equality and the rule holds in this case.

This method is true for both equal and unequal panels. With uniform loads the simplest way often is to draw the influence line for shear and compute its area, thus determining the actual value of the shear at once.

Approximate Method.—For uniformly distributed live loads the following approximate method is often used. It is somewhat

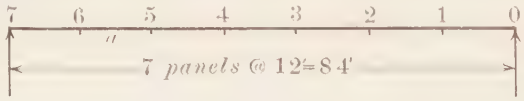


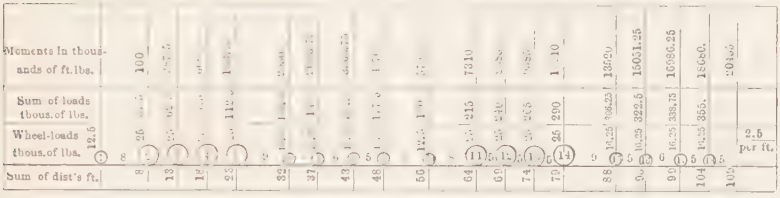
Fig. 13.

on the safe side. Compute a panel load. This will equal wp where w is the intensity of the load and p is the panel length. Apply this as a concentrated load at each panel point to the right or to the left of the one in which the shear is desired and compute the shear from these loads. Whether the loads are applied to the right or to the left of the panel will depend upon whether the maximum positive or negative shear is sought. This method is evidently in error on the safe side as a full load could not

obtain at one side of a panel with no load on the next panel point. This method would be applied as follows to find the maximum positive shear in panel a of the girder shown in Fig. 13. Uniformly distributed live load (assumed) $=2000$ lb. per foot. Panel load $=12\times2000=24{,}000$ lb.

$$\text{Max. shear in panel } a=\frac{1+2+3+4+5}{7}\times24{,}000=51{,}430 \text{ lb.}$$

Computations of Moments and Shears in Actual Case Using Moment Diagram.—The span chosen will be the one illustrated on Plate I which has four panels of 11 ft. 3 in. and the loading used will be Cooper's $E50$, the moment diagram for which is given


Moments in thousands of ft.lbs.
100
7010
13650
15051.25
16986.25
18660.
Sum of loads thous.of lbs.
215
290
Wheel-loads thous.of lbs.
12.5
26
25
26
25
26
25

about the right-hand support is readily found by adding to the moment of all the loads about load 7 (2693.75) the sum of all the loads 1 to 7 (145) multiplied by the distance 4.75 from load 7 to the support. That this is correct may be seen by considering that the moment of the resultant of all the loads on the span about the right-hand support is what is sought. The moment of this resultant about load 7 is evidently 2693.75. Its moment

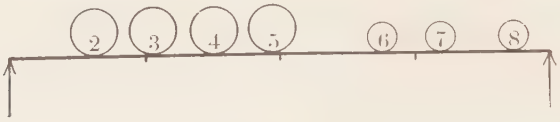


Fig. 16.

about the right-hand end is evidently found by considering that its arm is increased by 4.75 ft.

	2693.75	
$145 \times 4.75 =$	688.75	
	45)3382.50	Moment about right support.
	75.16	Total left-hand reaction.
$100 \div 11.25 =$	8.88	Left reaction on stringer.
	66.28	Max. live shear in end panel.

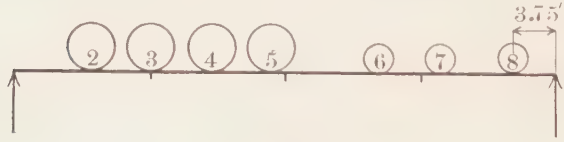


Fig. 17.

Fig. 17 shows a position of wheels which it is evident also satisfies the criterion for maximum shear. That it does so is because the first wheel goes off the span as the third one is moved up to the panel point. In an intermediate position as shown in Fig. 16 the sum of the loads on the panel is

$$25 \times 4 = 100 < 161.25 - 12.5 = 148.75$$

When the third wheel reaches the panel point we have

$$50 \times 4 = 200 > 148.75$$

which indicates a maximum.

We must now compute the shear with the wheels in this position and see how it compares with that already computed.

	3563.75	
$12.5 \times 43 =$	537.50	
	3026.25	Moment of loads on span about load 7.
$3.75 \times 148.75 =$	557.80	
	45)3584.05	
	79.67	
$\frac{5}{11.25} \times 25 \quad =$	11.11	
	68.56	Max. live shear in end panel.

It will be seen that this is greater than the amount 66.28 determined for the position of wheels shown in Fig. 15. There is a method of determining in advance with certainty which wheel

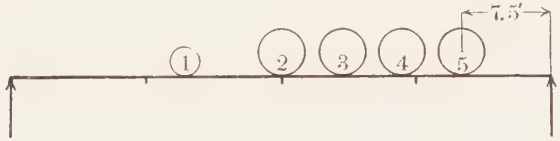


FIG. 18.

will give a maximum but it is so cumbrous and awkward that it is easier to compute an extra value for shear occasionally, as was necessary in this case. The live shear in the second panel is found similarly and all the necessary work is indicated below. Try the second load at the right-hand end of the second panel

$$37.5 \times 4 = 150 > 112.5$$

Therefore the shear will be a maximum with the wheels in position shown in Fig. 18.

	1037.5	Moment of all loads to left of 5 about 5.
$112.5 \times 7.5 =$	843.75	Sum of loads times distance from 5 to end of span.
	45)1881.25	Moment about right support.
	41.81	Left-hand reaction.
	8.88	Reaction on stringer at left of panel.
	32.93	Max. live shear in second panel.

For live moment at center we should have a heavy load at the center and other heavy loads near it. Try wheel 4 of engine at center (see Fig. 19). In finding the average loads to right and left of the section we can evidently choose any unit of length we please, such as a panel length, etc. In this case we will use half the span which will avoid any actual division.

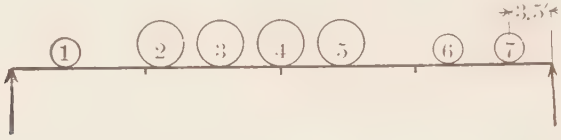


Fig. 19.

	Average to left		Average to right
Wheel 4 to right of section.....	62.5	<	82.5
Wheel 4 to left of section.....	87.5	>	67.5

Therefore wheel 4 gives a maximum. The reader should note that the drivers of the second engine often give a maximum moment at or near the center of a bridge. The moment with wheel 4 at the center is found as follows:

First find the moment of all the loads about the right reaction.

$$\begin{aligned} & 2693.75 \\ 145\times 3.5 = \ & 507.5 \\ & 3201.25 \end{aligned}$$

If this result be divided by 45 we will obtain the left-hand reaction. If the reaction be multiplied by 22.5 its moment about the center will be found. It will be noticed that these two processes combined are the same as dividing by 2.

2)3201.25

1600.625	Moment of left reaction about center.
600	Moment of loads to left of 4 about 4.
1000.625	Max. live moment at center.

The maximum live moment at the panel point next to the end may evidently be found by multiplying the maximum live shear in the end panel by a panel length.

$$68.56\times 11.25 = 771.3$$

This method can evidently only be used in the end panel.

CHAPTER II

RIVETS

General.—Structures are generally built up of many separate pieces which must be fastened together in order that they may act as a unit. This fastening is usually accomplished by means of rivets and pins. A rivet is a cylindrical piece of metal, made with a hemispherical head at one end larger in diameter than the body of the rivet. After insertion in holes previously made in the parts to be fastened together a head is formed on the other end either by pressure or by hammering. Rivets smaller than 1/2 in. in diameter are usually headed over or "driven" cold. Rivets of larger diameter are heated to a yellow heat and inserted in the parts and headed over before they have an opportunity to cool. When driven in this manner, the shank, or cylindrical portion, of the rivet expands under the pressure of the riveting machine until it fills the hole completely. As it cools, it contracts both in diameter and length. The diametral contraction is so small that it probably simply relieves the pressure without destroying the contact between the surface of the rivet and the hole. This pressure is caused by the rivet being squeezed by the machine in the process of forming the head. The longitudinal contraction of the rivet draws the parts closely together and holds them very firmly.

Action of a Rivet.—The action of rivets is not clearly understood and it is very difficult to determine experimentally. The theory on which the calculation of a rivet is based treats it as a closely fitting pin inserted in a hole, the heads being assumed to act merely to keep the pin from falling out. As a matter of fact when rivets are driven hot the heads draw the plates so tightly together that a great deal of friction is developed between the contact surfaces. It is probable that in many cases the friction between the surfaces keeps the plates in their relative positions rather than the pin-like action of the rivet. It is, however, the universal custom to consider that the rivet acts as a pin and the following treatment is based upon that theory.

Forms and Strengths of Joints.—The forms and combinations of forms of riveted joints are numerous and no attempt will be made to treat them all or even to enumerate them. The underlying principles will be developed, however, and the extension to cover cases other than those illustrated will be left to the student. We will first consider the simplest form of joint, known as the single riveted lap joint, as shown in Fig. 20. To be of good design a joint should be as nearly as possible of equal strength against all ways of failure. The joint shown may fail in any one of three principal ways. These are:

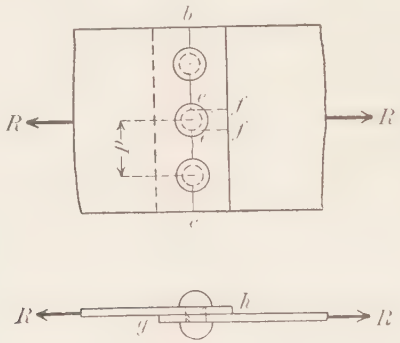


Fig. 20.

(1) Tearing along the line *bc*.

(2) Shearing of rivets on plane *gh*.

(3) Crushing of rivets or plates on their surfaces of contact which surfaces are indicated by cross-hatching in the figure.

There are other ways of failure such as

(4) Shearing out of plates on lines *ef*. This is easily guarded against by making the distance from center of rivet to edge of plate one and one-half rivet diameters or slightly more.

(5) Bending of rivets, a form of failure which is unusual because the rivet is prevented from bending by contact with the sides of the hole.

(6) Bending of plates in a joint of this form. This may be avoided by using another form of joint and is not often a cause of ultimate failure because the bending of the plates brings the forces R (Fig. 20) more nearly in line and reduces the couple causing the bending.

The resistance to failure in each of the first three ways is measured as follows:

(1) The applied load causes a tensile stress on an area found by multiplying the length *bc*, less the sum of the diameters of the rivet holes, by the thickness of the plate. It is this tensile stress which tends to tear the plate along the line *bc*.

(2) The applied load causes a shearing stress on the plane *gh* which is resisted by the cross-sectional area (circular) of the rivets.

(3) The applied load tends to cause the rivets or plates to crush on their area of contact. This crushing or compression is resisted by the area of contact. This area is always taken in computations as equal to the product of the diameter of the rivet and the thickness of the plate.

The theoretically perfect joint is one in which the strength in all ways is the same. The method of procedure in designing such a joint of the type shown in Fig. 20 will now be taken up.

Elements Affecting the Design of a Lap-joint. The first step in the design of a joint is to determine the size of rivets to be used. The second and third of the three principal ways of failure evidently depend on the size of the rivets. The ideal size of rivet from the standpoint of design is one in which the strengths in crushing and shearing are equal. It is seldom possible to use the exact size of rivet called for by these considerations because the numerical results of computations are generally in odd decimals of inches while plate thicknesses and rivet diameters used in structural work vary by sixteenths and eighths of an inch respectively. After having determined the rivet diameter the pitch or distance between centers of rivets must be determined. Knowing the stress to be carried per inch of width of plate, and knowing how much one rivet will carry, it is evidently easy to determine the pitch by dividing the latter by the former. The holes in the plate evidently weaken it and the result is that it must be made thicker than would otherwise be necessary in order to have enough area in the section taken through the rivet holes, on a plane passing through the axes of the rivets (*bc*, Fig. 20), to carry the tensile stress which the joint must bear. An arrangement of rivets, then, which brings a considerable number of rivet-holes close together in one row is evidently not desirable as it largely increases the required thickness of plate. As plates come in large sheets of uniform thickness and it is not practicable to make them thicker along the row of rivets than they are else-

where, anything which increases the thickness of plate operates against economy. On the other hand, anything which makes it possible to reduce the thickness of plates is desirable.

Efficiency of a Joint.—By the efficiency of a joint is meant the ratio of the least strength of the joint to the strength of the unpunched plate. It is evident that the thickness of metal required in any plate can be determined if the efficiency of the joint is known by dividing the total tensile stress to be carried by the allowable tensile fiber stress and by the efficiency of the joint.

Fiber Stresses.—The fiber stresses chosen have a very considerable effect upon the proportions of a joint. As a general proposition they should be so chosen that the factors of safety are the same against all three kinds of stress, viz., tension, compression and shear. The ultimate strength in tension is generally about two-thirds of that in bearing and the ultimate strength in shear is generally about two-thirds of that in tension. These proportions are by no means always adhered to.

Illustrative Problem.—Let us assume the following conditions: A plate of indefinite width is to carry a tensile stress of 2800 lb. per inch of width and is to be spliced by a single riveted lap-joint at some point in its length. The width of joint to be considered should be the distance p (Fig. 20) between centers of rivets. If we assume an efficiency of joint of 60 per cent. and the following fiber stresses

$$f_t = 16{,}000 \text{ lb. per square inch}$$
$$f_b = 24{,}000 \text{ lb. per square inch}$$
$$f_s = 11{,}000 \text{ lb. per square inch}$$

we will have a required thickness of plate of

$$\frac{2800p}{p \times 16000 \times 0.6} = 0.29 \text{ or } 5/16 \text{ in.}$$

The proper diameter of rivet will be determined by making the values of the rivet in shear and bearing equal. If d be the diameter of the rivet

$$0.7854d^2 \times 11{,}000 = 24{,}000 \times 5/16 \times d$$
$$d = 0.87 \text{ in. or } 7/8 \text{ in.}$$

The pitch p will be determined by making the strength of the plate between adjacent rivets equal to the strength of the rivet. The distance between adjacent rivets is determined by subtracting 1/8 in. more than the diameter of a rivet from the pitch p. The hole is made 1/16 in. larger than the rivet diameter, and 1/16 more is allowed for material around the hole which may be injured in punching the hole, etc. The strength of the length p in tension then must equal the strength of one rivet. The strength of the rivet in both bearing and shear must be calculated and the lesser value used because the diameter used is usually not exactly equal to that computed. The reason

for this is that computations result in odd decimals of inches, whereas rivet diameters vary by eighths of inches.

Strength in shear $= 0.7854 \times 0.875 \times 0.875 \times 11{,}000 = 6600$ lb.

Strength in bearing $= 24{,}000 \times 5/16 \times 0.875 = 6560$ lb.

$\{p - (0.875 + 0.125)\} \times 5/16 \times 16{,}000 = 6560$

$p - 1 = 1.312$

$p = 2.312$ in. or 2-5/16 in.

We must now check our assumed efficiency by calculating the ratio of the least strength of the joint to the strength of the unpunched plate. The strength of the unpunched plate is

$$2.3125 \times 5/16 \times 16{,}000 = 11{,}563 \text{ lb.}$$

and the efficiency is $\frac{6560}{11563} = 57$ per cent.

This value of the efficiency should be substituted for that assumed (60 per cent.) in the computation of the required thickness of plate and the thickness "t" recomputed.

$$t = \frac{2800}{16000 \times 0.57} = 0.306 \text{ or } 5/16 \text{ in. as before.}$$

The thickness of plate which would be required if it were possible to obtain a joint of 100 per cent. efficiency would be equal to $\frac{2800}{16000} = 3/16$ in. In other words, we must add 66-2/3 per cent. to the metal used in the plates throughout the whole length in order to overcome the inefficiency of the joint as designed. The next question is, How can we increase the efficiency of the joint so that it will not be necessary to use such a large excess of plate? If instead of arranging our rivets all in one row as shown in Fig. 20 we arrange them as shown in Fig. 21 we will largely increase the efficiency of the joint. We will assume to begin with that the two rows of rivets will be a sufficient distance g apart to avoid tearing on the diagonal lines shown. To accomplish this the sum of the diagonal distances r and s must be at least equal to p minus the diameter of one rivet hole. If the thickness of plate be left as before, p would be increased to 3.624 or 3-5/8 in. because the strength of two rivets instead of one could be counted upon in the length p. The efficiency of the joint would be increased because while two rivets may be counted upon in one length p only one rivet hole needs to be subtracted from the area of the plate because the rivets are in two different lines.

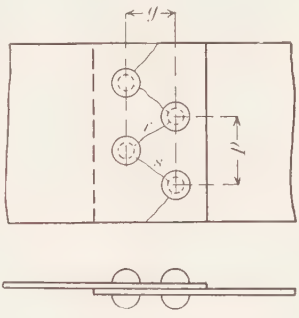


FIG. 21.

The strength of the unpunched plate then becomes $16{,}000 \times 5/16 \times 3.625 = 18{,}125$ lb. in a width p and the efficiency of the joint becomes

$$\frac{2 \times 6560}{18{,}125} = 72.5 \text{ per cent.}$$

The thickness of plate required with this efficiency of joint will be

$$t = \frac{2800}{16{,}000 \times 0.725} = 0.24 \text{ or } 1/4 \text{ in.}$$

This reduction in thickness of plate will result in a change in the diameter of the rivet from 7/8 in. to 0.694 or 11/16 in. This will result again in a change of pitch p from 3-5/8 ins. to 2-13/16 ins.

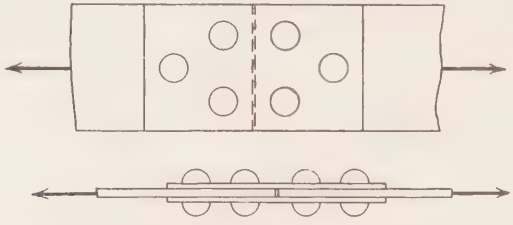


FIG. 22.

Strength in shear $= 0.37 \times 11{,}000 = 4070$ lb.
Strength in bearing $= 11/16 \times 1/4 \times 24{,}000 = 4125$ lb.
$\{p - (11/16 + 1/8)\} \times 1/4 \times 16000 = 4070 \times 2$
$p - 0.81 = 2.034$
$p = 2.844$ or 2-13/16 in.

The efficiency becomes $\frac{2 \times 4070}{16000 \times 0.25 \times 2.8125} = 72.4$ per cent. or practically that assumed.

A further increase in efficiency may be obtained by using a different form of joint as shown in Fig. 22. This joint brings the rivets into shear on two surfaces or into "double shear" as it is commonly termed, and has the effect of reducing the required diameter of rivet for a given set of conditions. The reduction in diameter operates to increase the efficiency of the joint as less material is cut from the plate.

The Structural Engineer's Method of Dealing with Rivets. – So far the subject has been developed rather from the standpoint of the builder of tanks and boilers than from that of the structural engineer. This method seems desirable as it gives the student a better idea of how a rivet acts than is obtainable by other kinds of treatment of the subject. The structural engineer generally uses a 7/8-in. rivet wherever possible and rarely any larger size although there is a tendency lately toward the use of larger rivets in some cases. This is because structural shops have been equipped with punches and riveters adapted to a rivet 7/8 in. in diameter or smaller. He further rarely mentions the "efficiency" of a joint, but usually simply puts enough 7/8-in. rivets in a connection to take care of the stress in the member. He considers

the "net area," which is the least cross-sectional area of the member through the most unfavorable set of rivet holes, and designs with that in mind. An illustration of the value of the boiler

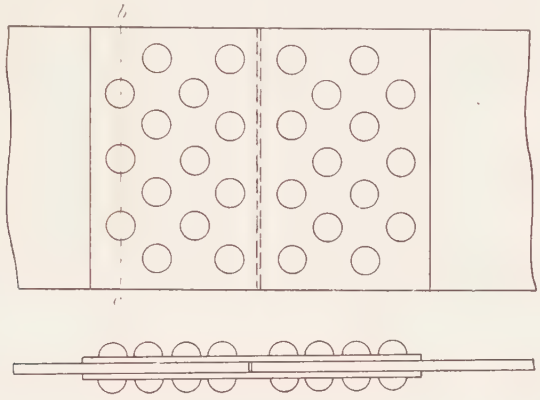


Fig. 23.

builder's viewpoint as opposed to that of the structural engineer will be introduced here. Suppose a flat bar is to be spliced. The average structural engineer would do it as shown in Fig. 23.

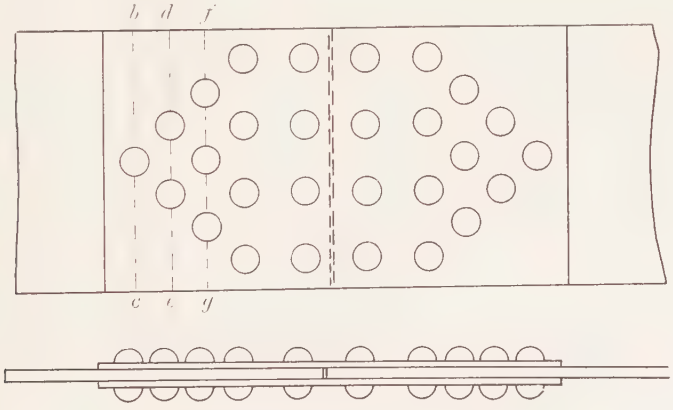


Fig. 24.

The "net section" will be that taken through or along the line *bc* and it will be noticed that three holes are to be taken out.

Suppose this joint be now designed from the boiler builder's standpoint; that is, with an eye to "efficiency." (See Fig. 24.)

Only one rivet is taken out on the section *bc*. The stress in the plate between *bc* and *de* is less than that to the left of *bc* by the value of one rivet, therefore two rivets may be taken out on the line *de*. The stress between *de* and *fg* is less than that to the left of *bc* by the value of three rivets, and so three rivets may be taken out on the section *fg* without impairing the efficiency of the joint, and so on. The splice plates will be a little longer and possibly thicker in the design shown in Fig. 24 than is required in the design shown in Fig. 23, but this will be more than offset because the net section of the main bar in Fig. 24 will be the gross area of the plate less one rivet hole, while in Fig. 23 three rivet holes must be subtracted. In the case shown in Fig. 23 there must be enough net area on the section *bc* to carry all of the stress in the main bar, which will require extra material equal to the cross-sectional area of three holes. This extra material will extend through the whole length of the piece. In the case shown in Fig. 24, the wasted material is represented by the cross-sectional area of one hole.

If there are holes which materially reduce the net area at some other point in the bar, it would be good economy to use enough rivets in the section *bc* to bring the net area down to that required at the other point. A question like this, however, must be settled on the merits of the particular case in hand. It should be understood that there are many cases of riveting, such as rivets connecting flange angles to webs, to which the efficiency principle is not applicable except so far as it may apply to the size of rivets used. In splices, however, and in connections of riveted trusses, it is quite important.

CHAPTER III

THEORY OF PLATE GIRDERS

Definition.—A plate girder is a beam composed of a deep thin web plate, and two flanges, which are generally composed of angles and plates, the whole forming what may be called a built-up I-beam.

Theory.—The common theory of beams may be applied to the plate girder and the stresses in the remotest fiber computed by the well-known beam formula $f = \frac{My}{I}$. This formula is, however, not a convenient one to apply in many cases, nor does it lend itself readily to purposes of design, as the computation of the moment of inertia of such a section is somewhat laborious. For purposes of design the following theory is much simpler to apply. It is not strictly correct, but gives results which are so nearly exact as to justify the theory in the majority of cases which are met with in practice. The degree of approximation involved in the theory depends entirely upon the geometrical properties of the cross-section of the girder. When a girder flange has a large number of cover plates piled upon it, or when the flange is of the shape of Fig. 59 *d* or *f*, there is considerable error in applying the approximate theory. It may be used in such cases to make a tentative design which is to be afterward checked over and corrected by the exact theory. The approximate theory referred to may be considered to be based upon the similarity of a plate girder to a truss having a solid web substituted for the diagonals and verticals. The top and bottom flanges of the girder take the place of the chords of the truss.

Let us consider now the section cut from a plate girder as shown in Fig. 25. From the theory of beams we know that there will, in general, be a shear and a moment on this section. This moment may be represented by a couple, Fh in the figure. If this were a truss, the points of application of the forces F composing the couple would be at the centers of gravity of the chords as shown in Fig. 26. Carrying out the analogy we arrive at

the conclusion that the two forces may be considered to be applied at the centers of gravity of the flanges of the girder. The stress in each flange is assumed to be distributed equally over it as would be assumed in the case of a truss. A little reflection will show that the approximate method gives a very ready means of designing a girder. The maximum loads, moments and shears are known in any actual case. The span is known and the depth of the girder is either known or may be assumed and the distance between the centers of gravity of the flanges, commonly called the effective depth, may then be closely estimated. For flanges of the usual form, Fig. 59, *a*, *b* and *c*, page 123, the effective depth may be assumed as from 1 to 2 inches

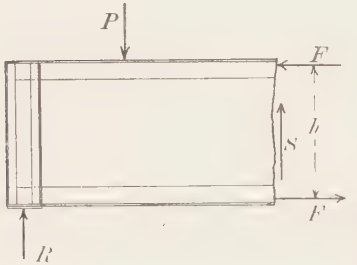


Fig. 25.

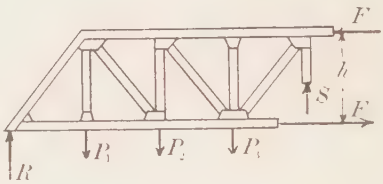


Fig. 26.

less than the depth of the girder. To find the resultant forces acting on the flanges at any section, we simply divide the moment existing at that section due to the external forces by the effective depth. If this force be now divided by the allowable fiber stresses in tension and compression, the required flange areas in the tension and compression flanges will result. After determining the required areas the composition of the flange should be determined. Generally at least one-half of the flange area should be directly riveted to the web. This means that in the ordinary form, one-half of the area will be in the angles and the other half in the cover plates. Generally the top and bottom flanges are made the same. The actual position of the center of gravity of the flange is then computed and the exact effective depth obtained. Using this exact effective depth the required flange areas can be recomputed and the design of the flanges revised if necessary.

The vertical or shearing stress on a section is assumed to be carried entirely by the web; and the web is proportioned to resist all of it, on the assumption that the shear is uniformly distributed over the cross-section of the web. The necessary web area can then be very quickly determined by dividing the maximum shearing force by the allowable shearing stress per square inch. It is evident that if the depth of the web is known or assumed, the thickness may be at once determined.

The above theory is simple, and easy of application. It depends upon various assumptions, and it is necessary in applying it to observe carefully certain principles in order to obtain satisfactory results. These assumptions and principles will now be given and should be read and thoroughly mastered before attempting to apply the theory.

Forces Applied at Centers of Gravity of Flanges.—This assumption would be correct if the stresses were uniformly distributed over the cross-section of the flange. We know, however, from the theory of beams, that such is not the case because the stress increases uniformly in intensity from the neutral axis to the outermost fiber of the beam. Consequently the resultant force on the flange will not act exactly at its center of gravity. The point at which it will act depends upon several conditions. In the first place the web plate carries some of the bending moment. No portion of the web plate, however, is considered in finding the position of the center of gravity of the flange, although it is quite usual to assume that a certain portion of the web may be counted upon as flange area.

To thoroughly understand the extent of the assumption we must compare the approximate and the exact formulas and see wherein they differ. It is easiest to do this by conceiving of a girder designed to resist a certain bending moment.

Let M = moment girder must resist.
I = moment of inertia of cross-section of girder.
y = distance to remotest fiber from neutral axis.
h = distance between centers of gravity of flanges.
A = total area of cross-section.
A_1 = area of flange.
f = maximum fiber stress caused by moment M.
r = radius of gyration of cross-section = $\sqrt{\frac{I}{A}}$.

When using the exact formula

$$f = \frac{My}{I} = \frac{My}{Ar^2}$$

When using the approximate formula

$$f = \frac{M}{hA_1}$$

r and $\frac{h}{2}$ are usually nearly equal because the moment of inertia of the web about its gravity axis and that of each flange about its own gravity axis is small compared to the area of the flanges multiplied by the square of their distance from the neutral axis of the section. Further, A is slightly greater than $2A_1$ although nearly equal to it. From the first formula we have then making these approximations

$$f = \frac{My}{2A_1\frac{h^2}{4}} = \frac{My}{A_1\frac{h^2}{2}}$$

The approximations so far are comparatively small in their effect upon the accuracy of the result obtained by using the approximate method. There is one last approximation which must be made to obtain the second formula from the first and that is to assume that $y = \frac{h}{2}$. We then have the second formula

$$f = \frac{M\frac{h}{2}}{A_1\frac{h^2}{2}} = \frac{M}{A_1 h}$$

The approximation which produces the greatest likelihood of inequality of results is the last one: that the distance to the remotest fiber equals half the distance between the centers of gravity of the flanges. The girders in which the greatest errors occur are those having a large number of cover plates piled on top of the flange angles and these having the type of flange shown in Fig. 59, *d*, *f*, and *g*. The percentage of error is somewhat less than would be indicated by taking the percentage of difference between the distances to the center of gravity of the flange and to the remotest fiber. It is evident that the error will be com-

paratively small for very deep girders and may be quite large for very shallow girders with a large number of cover plates on the flanges. Where it is necessary to use a large number of cover plates, an attempt is often made to reduce the error of the approximate method by arbitrarily providing that the distance between centers of gravity of the flanges shall be called no greater than the depth of the web even though it may be actually greater than that.

In such cases, and with flanges of the type of Fig. 59, *d* and *f*, the approximate method should be used to make a trial design

FIG. 27.

and then the final design should be made by using the exact formula $f = \frac{My}{I}$.

Fig. 27 is a photograph of the girder shown in section in Fig. 28.

Fig. 28 shows a very shallow and heavy girder. The location of the exact center of gravity of the flange stresses is shown together with the location of the center of gravity of the flanges computed in the ordinary way. It will be seen that they differ by a very small amount. This close agreement is due to omitting to take into account any portion of the web when computing the position of the center of gravity of the flanges. This omission, which is always made, tends to offset the omitting

to take any account of the fact that the fiber stress is greater at the outermost fiber than at the portions of the flange nearer the neutral axis.

In the girder shown in Fig. 28, the fiber stresses by the exact and approximate methods are found to be as follows for an assumed moment of 20,000,000 in.-lb.

$$f = \frac{My}{I} = \frac{20000000 \times 14.375}{19590} = 14{,}675 \text{ lb. per square inch exact.}$$

$$f = \frac{M}{A_1 h} = \frac{20000000}{22.12 \times 75.69} = 11{,}945 \text{ lb. per square inch approximate.}$$

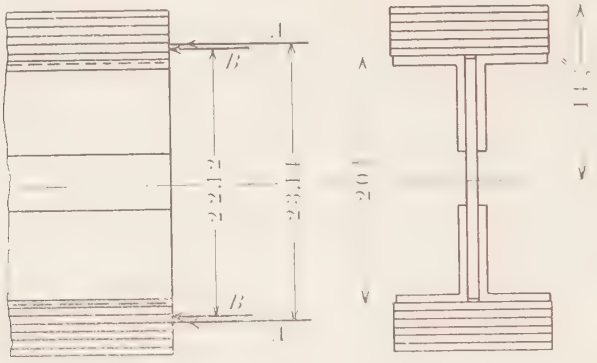


FIG. 28.—Arrows A indicate actual resultant stresses; arrows B point to centers of gravity of flanges. Composition of girder: one 20 in. $\times$ $\frac{7}{8}$ in. web; four 8 in. $\times$ 6 in. $\times$ $\frac{3}{4}$ in. angles; twelve 13 in. $\times$ $\frac{11}{16}$ in. cover plates.

The exact method gives a fiber stress in this case for a given moment 23 per cent. in excess of that given by the approximate method. It would not, then, be safe to design this girder by the approximate method. The distance from the neutral axis to the remotest fiber is seen to be 30 per cent. in excess of the distance to the center of gravity of the flange.

Effect of Location of Rivets on Distribution of Flange Stress. Another condition which affects the distribution of stress in the flanges is the location of the rivets connecting the flanges to the webs. These rivets transfer stress, which generally acts in a horizontal direction, from the web to the flanges. Of necessity, these rivets are generally located between the center of gravity of the flange and the neutral axis of the girder. Consequently there is some tendency to cause a moment in the flange due to the eccentricity of the rivets with reference to the center of

gravity of the flange. This moment tends in both top and bottom flanges to equalize the distribution of stress over the flange area. It is impossible to compute the exact effect of this condition. It should, however, be understood to exist. It is allowed for by so proportioning the flange that its center of gravity will fall at or inside of the edge of the web plate whenever possible. The idea in this is to keep the eccentricity of the rivets within limits. It also minimizes the errors inherent in the approximate theory.

Proper Distribution of Flange Area between Angles and Cover Plates.—The flange angles not only act as part of the flange area, but also serve to transfer stress from the web to the cover plates through the medium of the rivets. In order to perform this function properly, the angles should be about one-half the area of the flange. Interaction between these parts can best be understood by considering an extreme case. Fig. 29 is the cross-section of a plate girder taken from an English book on structures, and is representative of English practice. The horizontal shearing stresses are taken from the web by the flange angles and part of them are transferred to the cover plates. It must be borne in mind that the assumption underlying plate girder theory is that all parts of the flange are stressed equally. It is not, however, correct to assume that all parts of the flange will be stressed equally unless its construction is such as to insure an even distribution of stress. In the case illustrated in Fig. 29, stress is transmitted from the web to the flange angles and the proper proportion is presumed to reach the flange plates. Let us consider now what actually occurs. The stress is transmitted to the plates through the rivets *D* which are in the central portion of the plate. The result of this is that a large amount of stress needs to be transferred into the plates through these rivets and it must then distribute itself outwardly to the extreme edges of the plates in such a way that all parts of the plate will be equally stressed. What actually occurs is that the plate is much overstressed at the center and understressed at the edges, the distribution of stress over the plate being some-

FIG. 29.—Web 21 in. × ½ in.; flange angles 4 in. × 4 in. × ½ in.; cover plates 16 in. × ½ in.

what as shown Fig. 30, instead of being equal at all points as is assumed. The result of this is that the outer edges of the plate do not bear their fair share of the stress and the central portion together with the flange angles is considerably overstressed.

The point may be raised in the mind of the reader that so long as all parts of any given cross-section deflect an equal amount each part will carry its proper proportion of stress. In designs such as are shown in Fig. 29, all parts of the flange do not deflect equally but the cover plates assume some such shape as is shown in Fig. 31. A little consideration will show that when the girder is deflected the fibers in the part of the plate marked *A* will be shorter than those in the part marked *B* and consequently will carry less stress. Where several cover plates are used it is possible to stiffen them somewhat by adding a row of rivets outside of the flange angles. This general type of construction should be avoided.

Fig. 30.—Indicating roughly the distribution of stress in the cover plate of a girder of the type shown in Fig. 29.

The author has heard of one case in which a plate girder built along the general lines of Fig. 29 failed under loads which it should have supported without difficulty had it acted in accordance with the common theory. The distribution of area between the flange angles and plates was such, however, that the outer edges of the plates were carrying almost no stress and the angles were so overstressed that they tore in two, which naturally resulted in a progressive failure of the whole flange. It must be remembered that there are many assumptions made in plate girder design and it is necessary to understand them thoroughly and to understand the conditions under which they apply. It is further necessary to be sure that designs are so made that all the assumptions will be close to the truth. Consideration of what has been said in this paragraph leads to the conclusion that, when designing by the use of the ordinary plate girder theory, the cover plates should not be much wider than the total width of the flange angles connecting them to the web. Further, the

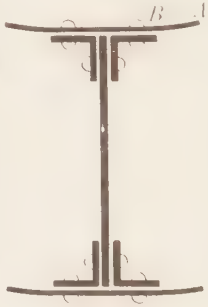


Fig. 31.

rivets connecting the angles to the cover-plates should be so located that each will distribute stress to its proper proportion of the width of the plate. This will lead to proportions about as shown in Fig. 32. Limitations of shop work and available sizes of material may prevent a strict adherence to this principle; but it should be adhered to as closely as such practical limitations will permit.

Portion of Web to be Considered as Flange Area. As stated before, if we consider the actual distribution of stress on the cross-section according to the beam theory, it will be seen that the web must resist some portion of the bending moment. The arrangement of the cross-section is such that the greater portion of the material is concentrated where the intensity of tensile or comparative stress is greatest; that is, at the extreme top or bottom of the girder. The web carries some of the bending moment and the question is, how much? When using the ordinary theory, the simplest method evidently is to find that portion of the area of the web which would, if concentrated in the flanges and regarded as flange area, carry the moment which the whole web actually would carry if analyzed according to the exact beam theory. To find that portion of the web, called the "web equivalent," which may be considered to be flange area we may proceed as follows:

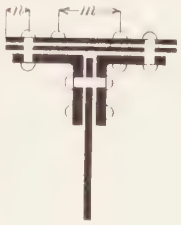


FIG. 32.—Distance m should not be more than 30 times the thickness of the outer plate. Distance n should not be more than 8 times the thickness of the outer plate nor more than 5 in.

We know from the beam formula that $M = \frac{fI}{y}$. The web, considered alone, is a rectangular beam. For a rectangular beam $\frac{I}{y} = \frac{1}{6}\ th^2$ in which t is the thickness of the web and h is its height. The moment which the web can carry then becomes $M = \frac{1}{6}\ fth^2$. If we consider h as approximately equal to the effective depth, which is very nearly true, we may write and interpret the formula as follows: $M = \frac{1}{6}\ fthh$. The quantity $\frac{1}{6}\ th$ is evidently an area. This area multiplied by f gives the total stress which the area bears. Multiplied again by the effective depth h it gives the moment exerted by a pair of such stressed areas which are located a distance h apart. The resultant of the whole stress in one flange is assumed to act at its center of

gravity and the centers of gravity of the two flanges are a distance h apart. We can apparently then conclude that 1/6 of the area of the web may be considered to act at the center of gravity of each flange. That is to say, 1/6 of the total area of the web may be considered as flange area in each flange. This value 1/6 is subject to some modification as it counts upon the gross area of the web, which is of course not available below the neutral axis because of the presence of vertical rows of rivets due to stiffener angles and web splices. If the web be considered to be perforated by 1 in. holes at intervals of 3 in., the moment of inertia will be about $\frac{1}{16}\ th^3$ instead of $\frac{1}{12}\ th^3$. As the proportion of the web to be included with the flanges varies directly as the denominator of this fraction, we should count upon approximately 1/8 of the web as available for flange area. This fraction 1/8 is not used by all engineers. Some count 1/12 of the web as available for flange area, claiming that it gives results which on the whole are better and that it is possible to secure better web splices, etc., where the fraction 1/12 is used than where 1/8 is used. The question seems to be not how much one wishes the web to be counted as carrying, but how much it actually does carry. Some engineers when designing assume that the web carries no moment at all. This results in an excess of area in the flanges, which is on the safe side. It is not fair to assume that the web carries no moment when checking over existing girders for strength. The author does not know that it is practicable to determine exactly which of these various assumptions is correct. He believes, however, that it is perfectly proper and safe to count upon one-eighth of the web as flange area. He does this in his own practice.

Gross and Net Areas of Flanges. In computing the area of the compression flange the gross or total area of each of the component parts of the flange should be used. In the tension flange the net area should be used. The same web equivalent is usually allowed in each flange. The net area may be defined as the area of metal which exists upon a plane perpendicular to the longitudinal axis of the flange and so located as to pass through as many rivet or other holes as possible. This net area should be used in proportioning or computing the strength of a tension flange. In general, when finding net areas of flange angles, the area of two rivet holes should be subtracted from each angle provided there are two rows of rivets in each leg. Otherwise

the area of one rivet hole should be subtracted from each angle. Cover plates should have two holes subtracted from them. Some authors claim that in finding the position of the center of gravity of a tension flange, the net area should be used, that is, that the possible effect of rivet or other holes on the position of the center of gravity should be computed. The author is unable to agree with this, but thinks that it is more nearly correct to compute the position of the center of gravity by using the gross area of the flange. His reason for this is that the net area exists at most over only a short portion of the flange, and between rivet holes the position of the center of gravity is certainly found by using the gross area. It is unlikely that the resultant flange stress shifts abruptly at rivet holes from the center of gravity of the gross area to the center of gravity of the net area. The truth of the matter probably is that the resultant stress acts at some intermediate point, probably nearer to the center of gravity which exists for the greater portion of the length of the girder, which would be that of the gross area of the flange. The difference in the position of the center of gravity as found by these two methods is so small that it is hardly worth considering. The point will be raised by some that using the net area to determine the center of gravity of the flange gives a less effective depth and consequently gives results which are on the safe side. This is true only in cases where cover plates are used. It seems that it is better to know what the assumptions are which are made, their effect upon the whole design, and within what limits the truth probably lies, than to merely make an assumption which is known to be safe. The approximate theory of plate girders gives as a rule results which are very closely correct in almost all cases, and it is not necessary to continually make assumptions which progressively put one further upon the safe side, if one thoroughly understands the subject and what he is doing.

Gross and Net Areas of the Web. —In proportioning the web it is proper to use the net area and it is customary to assume that this net area is three-quarters of the gross area. This value, three-fourths, is what may be called an average when based upon the usual pitch of vertical rows of rivets through the web. It is not the correct value to use in all cases. The proper one may vary either way from this. For instance, if a vertical row of one inch holes spaced three inches center to center exists throughout

the depth of the web plate at the point where the maximum shear acts, the proper value to use would be two-thirds. In this connection, in cases similar to that shown by the end stringers on Plate I, where the only stiffeners are those used to resist the reaction, some engineers claim that the gross area of the web may be counted as available for shear, and base their contention on the following reasoning: There being no vertical rows of rivets until the first reaction stiffener is reached, the gross area may be counted up to that point. At that point the rivets begin to remove stress from the web and remove it rapidly enough so that the intensity of shearing stress on the net area taken anywhere through the rivet holes will be no greater than that on the gross section of the beam.

Position of Neutral Axis.—It is often necessary to determine the position of the neutral axis and in this connection there is a diversity of opinion. Some authors claim that the neutral axis should be determined by considering the net section on the tension side and the gross section on the compression side. As stated in a previous paragraph, the net section exists only over a short proportion of the length of the beam and it seems very reasonable that the neutral axis should in general be nearer the position which is determined by using the gross area than that determined by using partly gross and partly net areas. It seems an entirely reasonable assumption that the axis does not shift violently up and down, but remains in substantially the same vertical position throughout the length of a properly designed beam. It seems reasonable that this position will be nearer to the neutral axis of the preponderating section, which is the gross section. The truth of the matter probably is that the neutral axis lies somewhere between the two extreme positions determined by the two methods mentioned above and probably nearer to that determined by using the gross section. It is not definitely known and cannot definitely be determined without extensive experiments just where the truth lies, either in this case or in the one treated in the second preceding paragraph. Such experiments as have been made, however, seem to bear out the author's position in the matter.

Web Stresses.—The exact theory of the distribution and kind of stresses existing in the web is quite complicated and is treated at length in the standard works on "Mechanics of Materials." We will, therefore, not go into the exact theory of web stresses,

but will give rather a brief outline of the stresses that exist and attempt to show how nearly our assumptions agree with the truth. In plate girder design it is customary, and nearly correct, to assume that the shearing stress is uniformly distributed over the web. The actual distribution is in all probability in reasonably strict accordance with the beam theory. The intensities of the horizontal and vertical shear at any point in the web are equal and are found by applying the formula from the "Mechanics of Materials"

$S = \frac{VQ}{Ib}$, where

S = the intensity of shear per square inch

V = the total vertical shear at section under consideration

Q = the statical moment of part beyond point at which it is desired to find the shear

I = the moment of inertia of the whole cross-section at section considered

b = width of section.

Tensile and compressive stresses also exist on the web, and acting at right angles to each other; their intensities at the neutral axis are equal to that of the shearing stresses. Above the neutral axis the compressive stress intensity increases while the tensile stress intensity decreases. Below the neutral axis the reverse holds true. A little consideration will show that inasmuch as the allowable intensity of tensile stress is larger than the allowable intensity of shearing stress, no difficulty need be anticipated from tensile stresses in the web so long as the shear is properly taken care of. The compressive stresses, however, need to be considered with some care as a slight distortion of the web from whatever cause (fabrication, blows during shipment or erection, etc.) may largely increase them through a species of columnar action. The result of this action in an extreme case would be to cause the web to buckle or wrinkle. Web stiffeners (see Plates I and III) are the means usually employed to prevent this tendency. The proper spacing to use for these stiffeners does not lend itself readily to theoretical treatment and there is a vast difference of opinion as to the proper way of spacing such stiffeners. This will now be discussed in detail.

There are several formulæ for the determination of the spacing of stiffeners. Several of these are given below: In all of them:

s = allowable shearing stress in web per linear inch of girder

in pounds. This stress equals the total shear divided by the effective depth at the point under consideration and will be recognized as a quantity found in determining the pitch of flange rivets by the approximate method.

t = thickness of web in inches.

d = least clear distance between flanges or stiffeners in inches.

$$s = \frac{12000\,t}{1+\frac{1}{3000}\frac{d^2}{t^2}} \qquad \text{Swain's formula.} \qquad (1)$$

$$s = \frac{16000\,t}{1+\frac{1}{3000}\frac{d^2}{t^2}} \qquad \text{A modification of the above.} \qquad (2)$$

$$s = 12000\,t - 40d \qquad \text{American Ry. Eng. Assoc.} \qquad (3)$$

$$s = 16000\,t - 120d \qquad \text{C. M. Spofford's formula} \qquad (4)$$

$$d = 60\,t \qquad \text{American Bridge Company} \qquad (5)$$

These formulæ give a wide variation in the spacing of stiffeners. For instance in the girder we have under consideration

$$s = \frac{142010}{66} = 2160.$$

The stiffener spacing from the various formulæ above is, in the case of the girder designed in Chapter IV, 22 in., 29 in., 57 in., 32 in., 22.5 in., respectively. The average value of all these results is 32.5 in. Of course the averaging of such a widely varying collection of results is a very unscientific way of reaching any conclusion as to the relative accuracy of any of the formulæ considered. Spofford's formula comes nearer to the average than any of the others.

A method of deriving two of these formulæ will now be given. Swain's formula may be deduced as follows: The intensities of tensile, shearing, and compressive stresses are the same at the neutral axis. The compressive stress at this point acts at 45 degrees with the horizontal and may be considered to act on a strip 1 in. wide, of the thickness of the plate, and of a length equal to the diagonal distance between flanges or between successive web stiffeners, whichever is the shorter. This strip may be considered as a column, and Rankine's formula for columns applied to it. The formula may be put in the following form:

$$\frac{P}{a} = \frac{f_c}{\left(1+\frac{1}{72000}\frac{l^2}{r^2}\right)}$$

In this formula and in the following discussion:

f_c = maximum compressive fibre stress existing on the most stressed section

E = modulus of elasticity of material

P = total load on column

A = area of cross-section of column

I = moment of inertia of cross-section A

l = length of column = $d\sqrt{2}$

r = least radius of gyration of column

f_s = shear per square inch of gross section of web.

The constant $\frac{1}{72000}$ is selected with reference to the fact that the column is more or less restrained by the tensile web stresses acting at right angles to the compressive stresses. The value of $\frac{P}{A}$ will equal, in this case, the shearing stress found by dividing the total shear on a cross-section by the gross area of the web or f_s.

Then
$$f_s = \frac{f_c}{1 + \left(\frac{1}{72000}\frac{l^2}{r^2}\right)}$$

For a rectangle of a width of one inch and a thickness, t, less than one inch $r^2 = \frac{t^2}{12}$.

Then
$$f_s = \frac{f_c}{1 + \left(\frac{1}{72000}\frac{2d^2}{\frac{t^2}{12}}\right)} = \frac{f_c}{1 + \left(\frac{1}{3000}\frac{d^2}{t^2}\right)}$$

If now we make f_c equal to the maximum allowable intensity of shearing stress on the cross-section of the web, which we will call 12000 lb. per square inch, we will have

$$f_s = \frac{12000}{1 + \left(\frac{1}{3000}\frac{d^2}{t^2}\right)}$$

$f_s \times t$ will equal the intensity of shear per inch of length of girder or s, and we may therefore write

$$s = \frac{12000\ t}{1 + \frac{1}{3000}\frac{d^2}{t^2}}$$

The next three formulæ are similarly deduced; (2) being derived at once from (1) by the substitution of a higher shearing value; (3) and (4) are deduced using the straight line column formula as a basis; and (5) may be deduced by the application of Euler's formula as follows:

Euler's formula may be applied to a strip similar to the one used in deriving Swain's formula. The form of Euler's formula will be that for a fixed ended column

$$P = \frac{4\pi^2 EI}{l^2}$$

$4\pi^2$ is practically equal to 40 and $E = 30{,}000{,}000$ for steel. We have then

$$P = \frac{40 \times 30{,}000{,}000 \times t^3}{2d^2 \times 12} = 50{,}000{,}000 \frac{t^3}{d^2}$$

P, the load on the column, may be considered to be equal to the allowable intensity of shearing stress multiplied by the cross-sectional area, A $(=t)$, of the strip under discussion. If the intensity of shear be taken as 10,000 lb. per square inch and if the load on the column be reduced to 72 per cent. of the value given by the above we will have

$$10{,}000t = 36{,}000{,}000 \frac{t^3}{d^2}$$

or

$$d^2 = 3600\ t^2$$

or

$$d = 60\ t$$

It must be borne in mind that Euler's formula gives the *ultimate* strength of the column. Reducing this to 72 per cent. of the value given by the formula allows a small factor of safety and at the same time makes some allowance for the stiffening effect of the tensile stresses which act along the sides of the strip of web under discussion. None of these formulas can be rigorously demonstrated on account of the many uncertainties in the case. The constants and allowances made are all matters of judgment; and in consequence many engineers prefer to use their judgment in the first place with regard to stiffener spacing.

The author's opinion is that the last formula (5) has much to commend it. The objection to it is that it gives a constant spacing of stiffeners throughout which is always the same for

every thickness of web, regardless of the fact that the actual intensity of shearing stress on the web may be very low because of the influence of other elements than shear in determining the web thickness. The following modification is therefore offered:

$$d = 60\, t \frac{S_a}{S_s} \tag{6}$$

where S_a = allowable intensity of shearing stress on web as determined by the specifications used.

S_s = intensity of shearing stress as determined by dividing the shear at the section where the stiffener spacing is desired by the web area.

In determining S_a and S_s, either gross or net areas of web may be used; but the gross area for one and the net area for

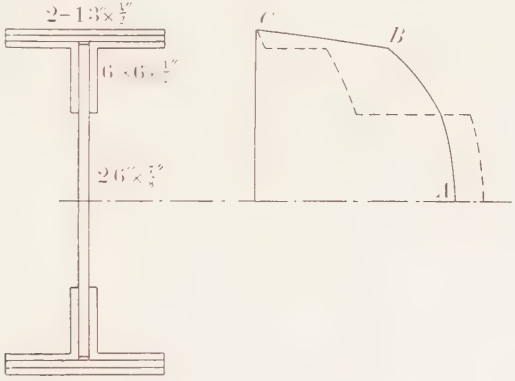


FIG. 33.

the other should not be used at the same time. This formula is easy to remember and simple to apply; and is probably as accurate in its results as any other.

In all of these formulas when d comes out as equal to or greater than the clear distance between flange angles it means that the web has sufficient stiffness in itself and stiffeners are not required.

As to the truth of the assumption that the web carries nearly the whole of the shearing stress, let us consider the typical cross-section of a girder as shown in Fig. 33. The intensity of shearing stress at any point per longitudinal inch of girder is given by the formula $S_1 = \frac{VQ}{I}$ where the quantities are the same as in the for-

mula given on page 43 except that both sides are multiplied by b and therefore S_1 = the shear per longitudinal inch of girder. In Fig. 33 the full line shows the shear per inch of length of girder and the dotted line shows the intensity in pounds per square inch at all points in the cross-section. By referring to the figure it will be seen that the quantity Q is very nearly constant between the points A and B because the statical moment of the flange is so much larger than that of the web. Above the point B the statical moment and with it the intensity of the shear decreases quite rapidly to zero at C. In this figure gross areas have been used to determine both the statical moments and the moment of inertia. It makes little difference in the result of computations of shear intensity whether the gross or net area is used in finding the necessary statical moments and moments of inertia. The gross area is much easier to handle. The maximum intensity of shear at the neutral axis of this girder (Fig. 33) is 11,800 lb. per square inch assuming a total shear of 250,000 lb. on the section. The intensity found by assuming that the shear is distributed uniformly over a net area equalling three-quarters of the gross area of the web is 14,700 lb. per square inch. This is evidently on the safe side.

Fig. 34.

Computation of Flange Rivets.—By flange rivets are meant rivets which connect the flanges to the web or which join the component parts of the flange together. In general, when flange rivets are spoken of, the rivets connecting the flange to the web are meant, although the term applies equally well to the other rivets in the flange. The theory of beams teaches us that there is a horizontal shear existing at every section of a beam subjected to shearing stress, and the computation of the required spacing of flange rivets is based upon this principle. By reference to Fig. 34 it will be seen that the flange rivets A are horizontal and that the plane upon which these rivets would shear is the surface of the web plate. Strictly speaking, this of course is not horizontal shear, but we assume that the usual formula for finding the intensity of the horizontal shear applies in such a case as this. This formula is $S_1 = \frac{VQ}{I}$ which has already been explained. (See page 48.) The portion whose static moment is found

consists of the angles and cover plates in one flange. I, the moment of inertia, is that of the whole cross-section of the girder at the section under consideration. For simplicity we will assume that both flanges are of the same composition which will locate the neutral axis at the center of the web. If d in the figure be the distance from the neutral axis to the center of gravity of the flange, the statical moment Q would equal the area of the flange times d. The moment of inertia of the whole cross-section would equal $2\ Ad^2$ + two times the moment of inertia of one flange about its own gravity axis + the moment of inertia of the web about the neutral axis. If these last two terms be neglected, the formula becomes $S_1 = VAd \div 2\ Ad^2 = \frac{V}{2d}$.

The quantity $2d$ in the above formula will be recognized as the effective depth.

Whenever the moment of inertia of each flange about its own gravity axis and the moment of inertia of the web about the neutral axis are small compared to the area of the flanges times the squares of their distances from the neutral axis, no great error will result from determining the horizontal shear per inch of length of girder at the point of attachment of the flanges to the web by dividing the total vertical shear existing at the section by the effective depth. Such error as there is in applying this method is on the safe side under such conditions. This may be seen by considering that the rivet spacing is found by dividing the value of one rivet by the horizontal shear per inch of length of girder. The rivet value is limited by either bearing on the web plate or double shear. The smaller the horizontal shear is, obviously the greater the rivet spacing will be, and by referring to the formula it will be seen that the larger the moment of inertia of the cross-section is, other things being equal, the smaller the shear per logitudinal inch of length will be. Therefore, any approximation which involves the assumption of a value of I which is smaller than the actual, will give a rivet spacing which is somewhat less than that which would be obtained by using the exact formula. The method of finding the horizontal shear by dividing the vertical shear by the effective depth is called the approximate method and is on the safe side for the reasons outlined above. It should only be applied in cases where the moment of inertia of the web plus the moment of inertia of the flanges about their own gravity axis is small

compared to the quantity obtained by multiplying the areas of the flanges by the squares of their distances from the neutral axis. A consideration of this condition brings the conclusion that the approximate formula is only applicable to girders having the form of flange shown in Fig. 59, *a*, *b* and *e*. For other forms of flanges, for instance those shown in Fig. 59, *c*, *d*, *f*, and *g*, where the moment of inertia of the flanges is known to be large about their own gravity axis, the exact formula should be used. Some engineers, when using the approximate formula, endeavor to compensate for the neglect of two of the terms entering into the exact determination of the moment of inertia by dividing the vertical shear by the depth of the web, or by the depth of the girder, instead of the effective depth. This gives a result which is more nearly in accord with that gained by using the exact formula than is obtained when the effective depth is used.

Variation of Section of Flanges. Because the ordinary plate girder flange is composed as a rule of several component parts, it is possible by omitting some of these parts at various points, depending upon the type of girder and kind of loading, etc., to make the actual section agree quite closely with that theoretically required. Consequently the metal is kept working up to somewhere near its full allowable working strength throughout. Methods of finding where to cut off different parts of the flange are given in Chapters IV and V. (See pages 109 and 136.) In dispensing with any portion of the flanges, there are two or three points which must be borne in mind. One is, that the flange must always be kept balanced about a vertical axis through the center of the web. This will be more clearly understood, perhaps, by reference to Fig. 59, *d* and *f*. Both plates (1) should be cut off at the same point which is another way of saying that one plate (1) should not be cut off and the other plate (1) carried on further. A little reflection will show that if these plates were cut at different points, the center of gravity of the flange would move to one side of the vertical axis through the center of the web. This would result in a lack of symmetry of the section about a vertical axis which would result in very undesirable and perhaps dangerous secondary stresses.

Another point to be kept in mind is, that portions of the top and bottom flanges should be dispensed with at the same point. These portions should be so chosen that the neutral axis will be

kept at approximately the same position throughout the length of the girder. It is very easy to do this with flanges of the form shown in Fig. 59, *b* and *c*. A plate may be cut off from the top flange and if a plate of the same width and thickness is at the same time cut off from the bottom flange, it is evident that the neutral axis will not be disturbed. The objection to the shifting of the neutral axis is that it causes secondary stresses which are difficult to compute and which can readily be avoided by proper design. Further consideration of this point leads to the commonly accepted principle of good design that the top and bottom flanges should be alike. It is evident that where a top flange of the form of Fig. 59, *f* is used, and a bottom flange of the form Fig. 59, *c*, a combination which may easily occur, the difficulty of keeping the neutral axis in the same general position, when portions of the flanges are cut off, is considerably enhanced, and one may be more or less hampered in his design by this consideration.

Stiffness and Deflection.—Plate girder bridges were for some time designed without any particular reference to their stiffness under loads. So long as they were carrying the loads, not much attention was paid to the amount of their deflection. It is coming to be recognized, however, that stiffness in a bridge is a very desirable element. With a stiff bridge, that is to say, one in which the deflection is comparatively small, there is less movement of the component parts, and consequently less tendency for the rivets to work loose. In a bridge which is not stiff under loads, the loosening of the rivets results in a considerable cost of maintenance; as it is necessary, at intervals depending upon the design of the details, to go over the bridge, cut out the loose rivets, and redrive them, a troublesome operation which should by all means be avoided. It is one which is very often neglected in highway bridges until it becomes absolutely necessary. Railroad bridges in the nature of the case are generally more carefully looked after. Designers at the present time are paying much attention to the stiffness of their bridges. The question of deflection is of considerable importance in connection with architectural work. It is generally assumed that a beam must not deflect more than 1/360 of its span. This ratio is required in a plastered ceiling to prevent the plaster from cracking. It should also be adhered to in other cases in architectural practice in order to prevent undue vibration or unsightly deflection. The loads under which the deflection is

computed should include both live and dead. For certain simple cases of loading, the amount of deflection is easily found, but for more complicated cases, the finding of the exact deflection may be a very laborious process involving not only tedious computations, but considerable liability to errors which are difficult to detect.

It is evident that if we can determine the limit of deflection in any particular case, our purpose will be served as well as though we determined the actual deflection. The following method of treatment was devised by W. H. Lawrence, professor of architectural engineering at the Massachusetts Institute of Technology, and is used here with his permission. The formulæ for deflection in four common cases are as follows: (For derivation see any standard work on applied mechanics.)

(1) Beam fixed at one end, loaded at the other;

$$d = \frac{1}{3}\frac{WL^3}{EI}$$

(2) Beam fixed at ends, loaded uniformly;

$$d = \frac{1}{8}\frac{WL^3}{EI}$$

(3) Beam supported at ends, load at middle;

$$d = \frac{1}{48}\frac{WL^3}{EI}$$

(4) Beam supported at ends, loaded uniformly;

$$d = \frac{5}{384}\frac{WL^3}{EI}$$

where d = maximum deflection of the beam.
W = total load.
L = span in inches.
E = modulus of elasticity of the material of which the beam is composed.
I = moment of inertia of the cross-section of beam.

These formulæ are correct for beams having a constant cross-section throughout their length and composed of homogeneous material. It is evident that none of these formulæ apply to beams irregularly loaded, such for instance as those shown in Fig. 35. In cases similar to these, the exact amount of the

deflection is nearly always immaterial. The following approximate method of finding the limit of deflection will give results which are sufficiently accurate for practical use with the error somewhat on the safe side.

For any given maximum fiber stress and loading, the ratio of deflection to span depends upon and varies directly with the ratio of the depth of the beam to the span. This may be proved as follows. A general formula for deflection may be written as follows:

$$d = \frac{CWL^3}{EI} \tag{1}$$

where the quantities are as heretofore indicated, and C is a constant depending upon the arrangement of the ends of the beam and upon the manner of loading.

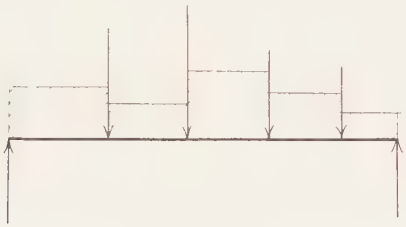


FIG. 35.

We know also that the moment M on a beam equals some constant C_1, multiplied by the total load on the beam and by its span. Written in algebraic language $M = C_1WL$. M also equals $\frac{fI}{y}$. We may substitute then in Equation 1, $\frac{fI}{y}$ for C_1WL. Equation 1, then becomes

$$d = C_2 \frac{fIL^2}{EIy} = C_2 \frac{fL^2}{Ey} \tag{2}$$

In this equation $C_2 = \frac{C}{C_1}$. If now, instead of expressing the deflection as a certain amount, we express it in terms of the span, we may write Z, the deflection in terms of the span, equals $\frac{d}{L}$. Dividing both sides of Equation 2, by L, we get

$$\frac{d}{L} = Z = \frac{C_2 fL}{Ey} \tag{3}$$

In this equation C_2 is a constant depending upon the manner of loading and

y = the distance to the remotest fiber from the neutral axis. In any given beam it is a quantity depending upon the geometrical properties of the cross-section. It is equal to half the depth of the beam where the beam has a horizontal axis of symmetry.

Any properly designed beam is working at its maximum fiber stress under the worst condition of loadiug. This is the same condition under which we have maximum deflection. We may say, therefore, that if we assume a constant maximum fiber stress, f, the deflection in terms of the span will depend upon the ratio of the span to the depth of the beam. Further, for any given manner of loading and given material, using a constant maximum fiber stress, we may design a series of beams whose ratio of deflection to span will be constant so long as we keep the ratio of the span to the depth of the beam constant. The use of Equation 3, as it stands is, however, by no means always simple or convenient, as it involves a knowledge of the constant C_2 which depends upon the manner of loading and is not easy to find in the general case. If we can find the limits of the constant C_2 under different conditions we will evidently have a means of finding the limit of deflection in any particular case. Ordinarily we do not care so much what the actual deflection is, if we know the amount which it cannot exceed, or in other words its maximum limit.

The case most commonly met is that of the beam supported at both ends; and we will proceed to investigate the effect of different methods of loading upon this type of beam. Suppose we assume a single concentrated load in any position as shown in Fig. 36, and let us further assume that this single concentrated load may be in any position on the beam and that it varies in amount as it moves in such a way as always to cause the maximum allowable fiber stress at the dangerous section of the beam. In order to keep the fiber stress constant as the load moves across the beam, the load must decrease in amount as it approaches the center, and will be a minimum when it reaches the center of the span. The deflection of the beam under such a loading will be very small when the load is just inside the support and will increase as the load moves toward the center of the beam. The maximum deflection which can be obtained by a

single concentrated load will occur when that load is at the center of the beam and can be found from the expression $d = \frac{1}{48}\frac{WL^3}{EI}$. (See Fig. 37.) Now let us assume that this load is gradually spread out from the center keeping it all the time uniformly distributed and varying its intensity so as to keep the maximum fiber stress a constant. (See Fig. 38.) During this change the

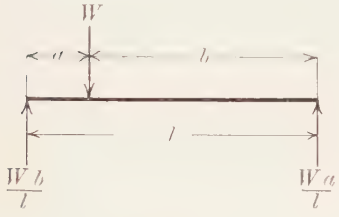


Fig. 36.

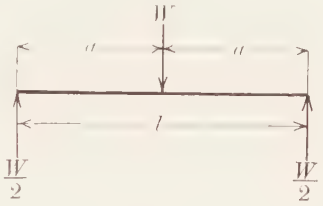


Fig. 37.

deflection will continue to increase until the load is uniformly distributed over the entire length of the beam. (See Fig. 39.) The deflection can then be found from the equation $d = \frac{5}{384}\frac{WL^3}{EI}$. This deflection is the maximum that can be produced by a single uniformly distributed load on any part of the beam. The point to keep in mind carefully in studying this treatment of deflection is that the maximum fiber stress at the dangerous

Fig. 38.

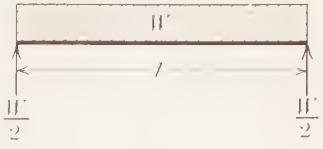


Fig. 39.

section remains a constant throughout all the changes in amount and position of load. Suppose now that this uniformly distributed load is split at the center and contracts toward the ends, still varying in amount so as to cause a constant maximum fiber stress. (See Fig. 40.) As the loads contract toward the end, the deflection increases still more and approaches a limit which will be the maximum deflection that can be caused by any manner of loading whatever. If we use two equal concentrated loads

symmetrically placed and moving toward the ends of the beam, the limit of deflection of the beam will be the same as before. (See Fig. 41.) It may be seen that the limit of deflection will be approached as the loads approach the end, by considering that the portion of the beam between the loads is exposed to a constant bending moment which produces the maximum allowable fiber stress on the remotest fiber at all points between the loads. This is evidently the condition which causes maximum deflection and its limit will be reached when the loads are very large and a very short (infinitesimal) distance from the ends of the beam.

As will be seen from the foregoing discussion, there are three important cases of loading illustrated by Figs. 37, 39 and 41 for which we must find the relations between depth and span.

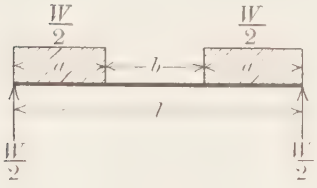


Fig. 40.

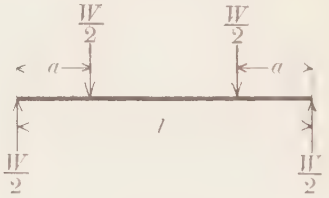


Fig. 41.

Case I. Fig. 37. Concentrated Center Loading $d = \frac{1}{48} \frac{WL^3}{EI}$. The maximum moment on such a beam equals $\frac{WL}{4}$. Then $C_1 = 1/4$

$$C = \frac{1}{48}$$

$$C_1 = 1/4$$

$$C_2 = \frac{C}{C_1} = \frac{1}{12}$$

We may read from Equation 3, $Z = \frac{1}{12} \frac{fL}{Ey}$. This is a general expression which covers all cases to which the beam theory is applicable, provided the proper values of y and f are used. In most cases y is equal to $\frac{h}{2}$ where h equals the depth of the beam.

Making this substitution for y, the equation becomes $Z=\frac{1}{6}\frac{fL}{Eh}$. This may be written $h=\frac{4}{24}\frac{fL}{EZ}$. This expression enables us to determine the required depth of beam for any given span, material, and deflection, with a concentrated center load. It is evident that the shape of the cross-section is immaterial so long as the neutral axis is half way between the top and bottom of the beam.

Case II. Fig. 39. A Beam with Uniformly Distributed Load.

In this case $C=\frac{5}{384}$

$$C_1=\frac{1}{8}$$

then $$C_2=\frac{5}{48}$$

and $$Z=\frac{5}{48}\frac{fL}{Ey}=\frac{5}{24}\frac{fL}{Eh}$$

$$h=\frac{5}{24}\frac{fL}{EZ}$$

Case III. Loading as shown in Fig. 41.

$$d=\frac{Wa}{48EI}(3L^2-4a^2) \qquad \text{(A)}$$

A direct solution is better in this case than an attempt to substitute in Equation 3. $M=\frac{Wa}{2}=\frac{fI}{y}$.

Substituting $\frac{fI}{y}$ for $\frac{Wa}{2}$ (Equation A), and substituting for y, $\frac{h}{2}$, we obtain

$$d=\frac{2f}{24Eh}(3L^2-4a^2)$$

or dividing both sides by L,

$$Z=\frac{f}{12}\frac{}{EhL}(3L^2-4a^2)$$

from this we obtain

$$h=\frac{f}{12}\frac{}{ELZ}(3L^2-4a^2)$$

In this case a is a variable which changes with the position of the loads. As a diminishes, the value of the equation becomes greater and the limiting value of h is found by making $a=0$, and equals $\frac{6}{24}\frac{Lf}{EZ}$

To summarize the results of this discussion, the maximum depth required by any single concentrated load is,

$$h=\frac{4}{24}\frac{Lf}{EZ} \qquad \text{(I)}$$

The maximum depth required by any single uniformly distributed load is,

$$h=\frac{5}{24}\frac{Lf}{EZ} \qquad \text{(II)}^{1}$$

The maximum depth required by any possible loading is,

$$h=\frac{6}{24}\frac{Lf}{EZ} \qquad \text{(III)}$$

These expressions give a ready means of determining the depth of beam required to keep the deflection down to any predetermined fraction of a span. Of these expressions, the first is rarely or never used in practice because it gives an error on the unsafe side on account of the omission of the weight of the beam. The error under these circumstances may be quite small but should be understood to exist. The second one is the one which covers nearly all cases of loading that occur in architectural practice. The limit of deflection for a bridge may also perhaps best be found by using the second expression.

The method of using these equations to determine the maximum limit of deflection in any case is plain. Their use in design is not quite so obvious. It is possible to determine beforehand the depth of beam which must be used to keep the ratio of deflection to span down to any predetermined amount, or to determine the allowable fiber stress for any given depth and deflection. This will now be illustrated by two problems.

Problem 1. Design a beam of hard pine of rectangular cross-section to carry the loads shown in Fig. 42. $f=1250$ lb. per

[1] This expression covers almost all cases of distributed and concentrated loads in combination, provided the center of the beam receives its fair share of loading.

square inch.; $E = 1{,}200{,}000$; limit of deflection $= 1/360$ of span.

In this case the second equation should be used

$$h = \frac{5}{24} \times \frac{1250 \times 18 \times 12 \times 360}{1200000} = 16.9 \text{ in.}$$

This result shows that the beam must have a depth of at least 16.9 in. in order not to deflect more than 1/360 of the span. The next larger market size is 18 in. The problem now becomes that of designing a beam 18 in. deep and strong enough to carry the load. The computation will be omitted but results in a beam 8 in. $\times$ 18 in.

It should be noticed that this method of first determining the depth of a rectangular beam and then finding the width necessary to give the required strength, is only applicable to isolated beams

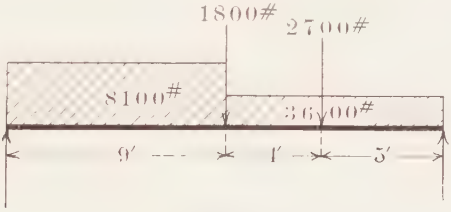


FIG. 12.

where the width may be altered without changing the load that the beam carries. It is not applicable to beams that are not isolated, as, for example, floor planking carrying a uniformly distributed load. In such a case as this the width of the plank cannot directly be changed without changing the load carried, and the fiber stress. Therefore, it is necessary first to assume the width and then find the required depth from the following expression which applies only to rectangular sections.

$$d = \frac{5}{384} \frac{WL^3}{EI} \quad I = \frac{bh^3}{12}$$

$$LZ = \frac{5WL^3 12}{384Ebh^3}$$

$$h^3 = \frac{5WL^2}{32EbZ}$$

Note that f is not determinate in this expression.

Problem 2. Design an I-beam to carry a 50-ton trolley car

over a 30 ft. span. The arrangement of wheels is as shown in Fig. 43. The dead load will be assumed as 300 lb. per foot per rail. For the sake of the problem we will say that the deepest I-beam readily obtainable is 24 in. and we will further apply ¶32 of the Specifications to this case. This paragraph states that rolled beams shall preferably have a depth of not less than one-twelfth of the span and that if shallower beams are used, the section shall be increased (*i.e.*, the fiber stress shall be reduced) so that the maximum deflection will not be greater than if the above limiting ratios had not been exceeded.

The limit of deflection for a beam having a depth of 30 in. (one-twelfth of the span), will first be found. Under the loading shown, the limit of deflection will be that given by applying Case II.

$$Z = \frac{5 \times 16000 \times 30 \times 12}{24 \times 30000000 \times 30} = \frac{1}{750}$$

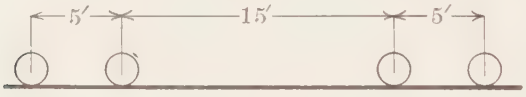


Fig. 43.

The problem now becomes that of choosing a fiber stress such that the deflection for a 24-in. beam shall not exceed 1/750 of the span.

Using again the equation of Case II, we have

$$24 = \frac{5}{24} \frac{f \times 30 \times 12 \times 750}{30000000}$$

from which $f = 12{,}800$ lb. per square inch.

We must then design a bridge capable of carrying the given loads using $f = 12{,}800$

Max. live moment = 159,700 ft.-lb. per rail.
Max. dead moment = $\frac{1}{2} \cdot 300 \times 30 \times 30 =$ 33,800 ft.-lb. per rail.
193,500 ft.-lb. total.

The required section modulus is $\frac{193500 \times 12}{12800} = 181.2$

This would require 1 24-in. 90 lb.-I per rail.

Illustrations like the foregoing could be multiplied ad infinitum, but the two just given should indicate quite clearly the application of this method. One other case ought to be men-

tioned and that is the solid or ballast floor bridges in which the floor may be very shallow and of considerable span. The author has in mind one case of a double track railroad bridge where the distance, center to center of trusses, is 32 ft. and the depth of the girders forming the floor is only 16 in. The limit of deflection in this case would be found by using the equation III because the loads are applied well toward the ends of the beams. In this particular case, the deflection is quite perceptible to the eye, which is not at all desirable, and it is of such an amount that there is liability of trouble from the working loose of rivets which connect the floor to the trusses. In this case a very low fiber stress and consequently a very heavy section would be required to keep the deflection within proper bounds. It is a construction which should be avoided wherever possible.

CHAPTER IV

DESIGN OF THROUGH PLATE GIRDER

Span and Type.—In this chapter it is the intention to design a through plate girder bridge of 45-ft. span and four panels with a so-called open floor, and to give the principles which should be followed in order that it may be constructed readily and economically so far as shopwork and erection are concerned. The live moments, shears, etc., at various points in the bridge have been predetermined. (See pages 19 *et seq.*) Plate I is a drawing of the complete bridge.

Loads.—In finding these stresses the loading known as Cooper's *E* 50 has been used for the live load. Various tables of dead weights have been constructed from time to time, but the writer does not know of any which fit the case under consideration closely enough to warrant their use. In fact, in most plate girder work it is possible to assume the dead weight of the girder itself with sufficient accuracy to make it unnecessary to use any tables. In any case, the weight of the girder itself should be computed when the design is completed in order to see whether it agrees with the assumed amount. If it does not agree, it is, in general, a very simple matter to make the necessary correction and an error does not as a rule affect the design appreciably in plate girder work. The reason for this is that generally plate girders are so short that the external loads upon them produce much greater stresses than their own weight does. Consequently, an error in the assumption of the dead weight does not so vitally affect the design as it does in longer structures of a different type, such as trusses, etc. For designing purposes the corrections for the dead weight are so easily made that no attempt will be made here to give any tables or rules for estimating the dead weight. It should be understood that such tables are of little real value in making designs but are principally useful for making approximate estimates of the comparative cost of structures of various spans.

We will adopt the plan of computing the dead weights as we proceed with the design. A summary of the stresses is given below. It is necessary to assume the dead weight at first, so two

columns are provided in the summary, one for assumed and the other for actual loads; assumed dead loads are to be put in the last column and the first column is to be filled out when the loads are determined after the structure is designed.

45 FT. THROUGH PLATE GIRDER

Summary of Moments and Shears.

STRINGERS

At end		Actual	Assumed
1. Max. live shear,		41,600 lb.	.....
2. Max. impact shear,	$\frac{300}{300+11.25}=96.5\%$	40,100 lb.	.....
3. Dead shear,		*1,700 lb.	1,700 lb.
Total shear at end.....		83,400 lb.	83,400 lb.

At quarter-point			
1. Max. live shear,		26,400 lb.	.....
2. Max. impact shear,	$\frac{300}{300+8.45}=97\%$	5,600 lb.	.....
3. Dead shear,		*850 lb.	850 lb.
Total shear at quarter-point.....		52,850 lb.	52,850 lb.

1. Max. live moment,		86,000 ft.-lb.	.....
2. Max. impact moment,	96.5%	83,000 ft.-lb.	.....
3. Max. dead moment,		*5,435 ft.-lb.	4,750 ft.-lb.
Total moment.....		174,435 ft.-lb.	173,750 ft.-lb.

FLOOR BEAMS

1. Max. live shear at end,		55,500	.....
2. Max. impact shear at end,	$\frac{300}{322.5}=93\%$	51,600	.....
3. Dead shear at end,		*4,670	5,075
Total shear at end.....		111,770 lb.	112,175 lb.

1. Max. live moment,		208,000 ft.-lb.	.....
2. Max. impact moment,	93%	194,000 ft.-lb.	.....
3. Dead moment at center,		*17,200 ft.-lb.	18,650 ft.-lb.
Total moment at center.....		419,200 ft.-lb.	420,650 ft.-lb.

45 FT. THROUGH PLATE GIRDER

GIRDER

Panel A		Actual	Assumed
1. Max. live shear,		68,560 lb.	.....
2. Max. impact shear,	$\frac{300}{345}=87\%$	59,650 lb.	.....
3. Dead shear at left end of panel,		*14,308 lb.	13,800 lb.
Max. shear panel A.....		142,518 lb.	142,010 lb.
Panel B			
1. Max. live shear,		32,900 lb.	.....
2. Max. impact shear,	$\frac{300}{333.75}=90\%$	29,600 lb.	.....
3. Dead shear at left end of panel,		*5,935 lb.	5,725 lb.
Max. shear panel B.....		68,435 lb.	68,225 lb.

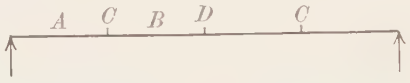


FIG. 44.

Panel point D			
1. Max. live moment,		1,000,000 ft.-lb.	.....
2. Max. impact moment,	87%	870,000 ft.-lb.	.....
3. Dead moment,		*188,400 ft.-lb.	181,750 ft.-lb.
Max. moment at panel point D...		2,058,400 ft.-lb.	2,051,750 ft.-lb.
Panel point C			
1. Max. live moment,		771,300 ft.-lb.	.....
2. Max. impact moment,	87%	671,000 ft.-lb.	.....
3. Dead moment,		*141,300 ft.-lb.	136,300 ft.-lb.
Max. moment at panel point C...		1,583,600 ft.-lb.	1,578,600 ft.-lb.

* Quantities marked with the asterisk are computed from known weights after structure is designed.

Specifications.—In what follows the reference "Spec." followed by a number, refers to the corresponding paragraph in the Specifications of the New York, New Haven and Hartford Railroad Company, dated 1912, which are reprinted herein on pages 184 to 212. These specifications are used also by the Boston and Maine, and Maine Central Railroads.

Arrangement of Computations.—The form and arrangement

of computations should be similar to that in these pages and students are particularly cautioned against slovenly or careless methods of setting down their work.

THE TIES

Loads and Design.—The ties should be designed first for bending and investigated for shear. The wheel load will be considered as distributed over three ties by the rails. See Spec. ¶ 9. This will give a loading on the ties as shown in Fig. 45.

It is unnecessary to consider dead load in this case as it is such a small proportion of the live load as to be negligible. The imact allowance is 100 per cent.

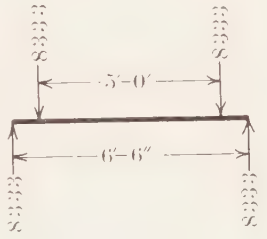


FIG. 45.

$$2\times 8333\times 9=\frac{1}{6}\times 2000\times bh^2$$

$$450=bh^2$$

where b is the breadth and h is the depth of a tie. The standard sizes of ties are 6×8, 7×9, 9×10, and 10×12 in.

For 6×8 $bh^2=384$ which is too small.

For 7×9 $bh^2=567$ which is sufficient.

The fiber stress of 2000 which we used in this specification is higher than that ordinarily specified for wood. The reason for this is that wood is better capable of resisting impact than is steel. In fact, it has been found by experiment that a wooden stringer will endure a greater deflection without injury under a suddenly applied load, than it will under a static load. As, however, specifications require the use of an impact allowance throughout, and as the use of the usual fiber stress for wood together with the impact allowance would give results unnecessarily far on the safe side; the allowable fiber stress is arbitrarily increased in order to make allowance for the impact percentage, which is too large for wood, although it is reasonable for steel.

Horizontal Shear.—The maximum intensity of horizontal shear needs investigation next. Using live load only and the formula for maximum intensity of horizontal shear on a beam of rectangular cross-section $s=\frac{3}{2}\frac{V}{A}$

where s = intensity of shear per square inch.
V = total vertical shear.
A = area of cross-section of tie.

$$s = \frac{3 \times 8333}{2 \times 9 \times 7} = 198 \text{ lb. per square inch (no impact).}$$

This value is much higher than that generally allowed; but as this intensity is only operative through 18 in. of the 10 ft. of length of tie (see Fig. 45), and as the intensity through the remainder is zero, it may well be considered that the reinforcing effect of the rest of the tie will prevent shearing along the neutral axis. It should be noted that the 9 in. is measured from center of rail to center of stringer and that the actual distance between inner edge of stringer and outer edge of rail is much less than this. The author has examined a considerable number of railroad bridges in the course of his professional practice and has so far not seen a tie which has failed in horizontal shear. The ordinary bridge tie may be considered to be entirely safe from horizontal shear.

THE STRINGERS

Width of Flange.—It is necessary for the width of stringer flange to be as much as the width of base of rail or width of tie-plate in order that the tie may not crush on top of the stringer. This consideration often precludes the use of I-beam stringers. If the two bearings (*i.e.*, on rail and stringer) are of equal width, the tie will cut under the rail first, because the load is more directly applied there and because water will not drain away from there as readily as from the surface between the tie and stringer. The minimum width of bearing surface between tie and stringer may be computed as follows: The allowable bearing pressure on this surface may be taken as 260 lb. per square inch without impact.

$$\frac{8333}{7 \times 260} = 4.57 \text{ in. required width of flange of stringer.}$$

The principles involved in the remarks made concerning the allowable fiber stresses in wooden ties and stringers apply here also.

Fastening Ties to Stringers.—The ties are usually notched to a depth of 1 in. where they rest on the stringers in order to

secure them against lateral motion. The C. M. & St. P. R. R. allows the web to project above the flange angles and makes a narrow slot in the tie so that the surface of the tie may rest directly on top of the flange angles. This has the advantage that the slots do not have to be carefully smoothed out to secure a good bearing as is the case with the ordinary construction. The tie rests on a sawed outer surface which probably secures for it a better bearing on the stringer than is obtained where a wide notch has to be cut smoothly. See Fig. 46 for the two methods of notching the ties. In both cases the ties should be held down by hook-bolts which should pass through every third tie (see Plate I).

Fig. 46.

Depth of Stringers.—The next members to design are the stringers. It should be noted that the depth of the floor is determined in any practical case by the elevation of the rails on the bridge and by the under-clearance. The floor includes rails, ties, stringers and floor-beams. The under-clearance may be defined as the line below which the bridge must not project. The only reason for building a "through" plate-girder bridge is because the difference in elevation of the rail and the under-clearance line is so small that a deck plate girder would be so shallow in proportion to its length as to require an extremely heavy and uneconomical section. As the available height for constructing a deck bridge diminishes, a point will evidently be reached where it will become more economical to use stringers and floor-beams whose function is to transfer the loads to a point sufficiently far distant from the track on either side to allow girders to be used which may project a sufficient distance above the rail to secure a reasonably economical depth. In general, the depth of a plate-girder should not be much less than one-twelfth of its span. In giving the data to his classes, the author has found it undesirable to give the students fixed clear-

ance lines to work to and has instead generally fixed the depth of the stringer as about one-sixth of its span. A sufficient thickness of web should be used in the stringers in all ordinary cases to render intermediate web stiffeners unnecessary. Therefore, the depth of stringer which will require the minimum amount of material is the depth at which the smallest permissible flange angle will be of just sufficient area in combination with the web to carry the flange stresses. (For further remarks on this economic depth, see page 97.) The minimum depth of stringer is generally determined, not as might at first be supposed by the maximum size of flange angle which is obtainable, but by the possibility of obtaining a sufficient number of rivets in the end connection to transmit the shear from the stringer to the hitch-angles connecting it to the floor-beam, or it may be limited by the minimum allowable pitch of flange rivets. It should be noted that where the floor must of necessity be very shallow, three or four stringers may be used under one track. These stringers should be so placed transversely to the track that they will be equally loaded. Fig. 47 illustrates the proper spacing to use for three stringers and a good spacing to use for four stringers. Three stringers are rarely used, but four stringers are quite common, especially where an *E* 60 loading is used together with a very shallow floor.

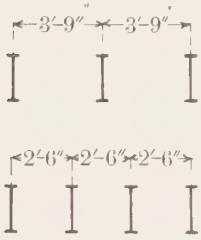


Fig. 47.

For the design in hand the dead weight of one stringer may be assumed as 100 lb. per foot. The weight of track should be taken as 400 lb. per foot. This includes track, commonly called "stock," rails, guard rails, guard timbers, ties, and the necessary spikes and bolts. The total assumed dead load on one stringer then equals $11.25 \times 300 = 3375$ lb.

The maximum dead moment then equals

$$1/8 \times 3375 \times 11.25 = 4750 \text{ ft.-lb.}$$

Dead shear at end $1/2 \times 3375 = 1700$ lb. nearly.
Dead shear at quarter point $1/2 \times 1700 = 850$ lb.
These quantities should now be put in the "assumed" column in the summary. In the case we have, we will assume the stringer equal in depth to one-sixth the span or say 24 in. This proportion will in general give a good design. In this connection

it seems desirable to show the possible influence of the details of the end connection upon the depth of the stringers.

Influence of End Connections on Depth of Stringer. The detail shown in Fig. 48 would require a depth of stringer which may be determined as follows:

Maximum end shear on the stringer from the summary (page 63) is 83,400 lb.

The value of one 7/8-in. shop rivet in single shear is

$$12{,}000 \times 0.6 = 7200 \text{ lb.}$$

(Spec. ¶'s 21, 22 and 28.)

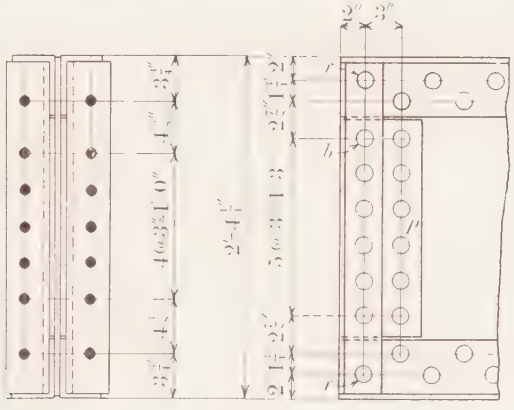


Fig. 48.

Number of rivets required to connect hitch (often called clip) angle to stringer is

$$\frac{83400}{2 \times 7200} = 6 \text{ rivets in double shear.}$$

The bearing value of the rivets will in almost all cases be greater than this as a tight filler will almost invariably be used in this type of connection. A tight filler is a plate which is used between the flange angles to fill the space under a hitch or stiffener angle, and which has rivets passing through it outside of the hitch or stiffener angle. The plate *p*, Fig. 48, is a tight filler. By referring to Fig. 48 it will be seen that it is possible to obtain as many rivets in the tight filler, in addition to those which pass through the stiffener angle, as there are rivets in the stiffener

angle. In this case there will be room for 12 rivets in all. In general the two rivets r which pass through the hitch angles and the flange angles of the stringer should not be counted as connecting the hitch angles to the stringer because these rivets are at the point where the maximum shear on the stringer occurs and hence are fully stressed in fulfilling their function as flange rivets. The total depth of stringer with this type of connection then would be as follows:

(1) $2\times5=10$ in. width of two flange angles assuming them as 5 in.

(2) $2\times1\text{-}1/2=3$ in. space between flange angles and rivets nearest flange angles passing through hitch angles.

(3) $5\times3=15$ in. 5 spaces of 3 in. between the six required rivets.

28 in. total depth.

Item (1) requires some explanation. In bridges of this character and designed for this loading, the required spacing of flange rivets is too small to permit of the use of a single row. It is, therefore, necessary to use an angle leg of sufficient width against the web to allow the use of two rows of staggered rivets. The smallest angle leg which will admit two rows of staggered rivets is a 5-in. one.

FIG. 49.— Effect of punching hole too close to edge of piece.

Item (2) also requires some explanation. The rivet hole in the filler for the rivet b must not be closer to the edge of the filler than 1-1/2 in. The reason for this is that if the hole be punched closer than this to the edge of the plate, the material between the hole and the edge will be so injured as to be of doubtful strength and may even be bulged out (see Fig. 49).

This bulging not only is a sign of injury to the plate but might cause difficulty in properly inserting the filler between the flange angles. The proper distance from center of hole to edge of plate for different sizes of rivets has been determined in practice from experience and is given in most specifications as the following values:

For 7/8-in. rivets 1-1/2 in. for sheared edges, and 1-1/4 in. for rolled edges.

For 3/4-in. rivets 1-1/4 in. for sheared edges, and 1-1/8 in. for rolled edges.

This filler must be made tight by means of extra rivets as it is not considered good practice to count rivets extending through a loose filler at their full value. The reason for this is that a rivet passing through a loose filler is supposed to be exposed to some bending. The presence of the loose filler separates the plates between which the rivet is transferring stress and consequently a bending action is set up.

An arbitrary increase in the number is usually made under such circumstances. In this case the allowance would be 50 per cent. (Spec. ¶ 60).

This would lead to an impracticable depth of member and consequently we will use a tight filler as shown in Fig. 48. The

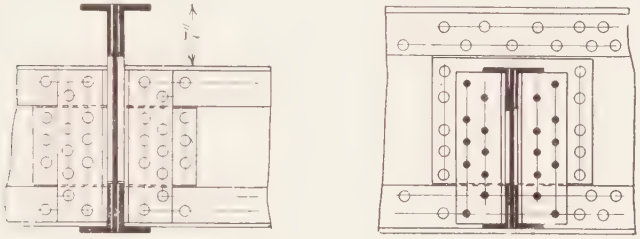


Fig. 50.—Type of connection of stringers to floor-beams used where floor must be shallow.

rivets through the tight filler and web are spaced opposite those in the flange angle as a rule. This commonly gives more rivets than computation shows to be necessary.

The depth of stringer necessary for this type of connection would then be 28 in. The minimum depth of stringer which can be used may be determined similarly. In this case a wide legged angle must be used for the hitch angle as shown in Fig. 50. The minimum possible rivet spacing for a 6-in. angle may be determined from the tables on pages 274–5. The gages for a 6-in. leg are 2-1/2 in. and 2-1/4 in. The latter is the one we shall use. The minimum distance center to center of rivets as allowed by the specifications (¶ 41) is 3.06 in. or possibly 3 in. Looking now at the table giving distance center to center of staggered rivets we find that for $a = 2\ 1/4$ in. and $x = 3$ in., the value of $b = 2$ in. We can, therefore, use a spacing of 2 in. Another element must, however, be considered and that is clearance for driving. In order to be driven by a machine a radius varying

according to the size of the rivet must be left clear all around it. This radius or imaginary cylinder must not be encroached upon at any point; for a 7/8-in. rivet, it is 1-1/4 in. Assuming a 9/16-in. thickness of hitch angle $c = 2\text{-}1/2 - 9/16 = 1\text{-}15/16$ in., which gives ample room for machine driving (see table on page 275, entitled "Least Stagger for Rivets"). For further remarks in this connection the reader is referred to Chapter VII on shop practice, page 172.

The thickness of the hitch angle is assumed as 9/16 in. to allow for planing or facing off the ends in order to make the stringers of exactly the right length. A little reflection will show the great importance of having the stringers of exactly the right length as otherwise the floor-beams would be buckled when the bridge was erected. The facing of the ends of the stringers to exact length is required by most roads at the present time. (Spec. ¶ 70, 145.) The minimum depth then is

$2 \times 5 = 10$ in. as before.
$2 \times 1\text{-}1/2 = 3$ in. as before.
5×2 in. $= 10$ in.—5 spaces of 2 in. between rivets.
23 in.

If it happened to be necessary to use a less depth of stringer than the above, it would be necessary on account of the end connection to use three or more stringers. In those rather rare cases where one is free to choose the depth of stringer or girder without clearance restrictions, it is possible to use the economic depth.

Design of Stringer Web.—We will proceed with the design of the stringer having determined, in this case arbitrarily, that it will be 24 in. in depth. The web plate itself should always be made an integral number of inches deep and should never be given a depth involving fractions of inches. The reason for this is that the plates of fractional depth must be rolled to order, or else they will have to be sheared from the next widest plate in stock. Universal rolled (which means rolled on both sides and both edges) plates of ordinary width may be secured at any time in widths advancing by integral inches. It is desirable to have both edges of the plate rolled as it gives a better finished job. A sheared edge is also likely to have somewhat of a burr or rough edge on it which may project sufficiently to interfere

with the close fit of the flange angles. The thickness of the web must now be determined. Four things may influence us in this:

(1) Shear on the web.

(2) The fact that there must be enough bearing area between the web and the rivets connecting it to the hitch angle and its tight filler to transfer the maximum shear from the web.

(3) The web must be thick enough to make it possible to transfer the horizontal shear between the flange rivets and web with a practicable rivet spacing.

(4) The web should be thick enough to make web stiffeners unnecessary.

The first element is the one on which most books on design lay the greatest stress; the other three are not often mentioned. They are often, however, the elements which determine the thickness of the stringer web. We will discuss them in order.

(1) The minimum thickness of web must not be less than enough to give the area necessary to carry the maximum shear. The specifications we use, give the allowable shearing stress on webs (Spec. ¶ 21) as 10,000 lb. per square inch on the gross area for plate girder webs. Many specifications give a value for shear based on net area. It is practically the universal practice to consider the net area of a plate girder web as three-fourths of its gross area. To get this result a row of 3/4-in. rivet holes spaced 3 in. on centers extending throughout the depth of the web is assumed. Bearing this in mind, it is evidently entirely immaterial whether we take our working fiber stress in shear as 13,333 lbs. per square inch on the *net* area and consider the net area as three-fourths of the gross or take our working fiber stress as 10,000 pounds per square inch on the *gross* area. Evidently the latter method is simpler of application and as accurate when confined to plate girder webs as is the other.

$$\text{Web thickness required } \frac{83400}{10000\times 24} = 0.347 \text{ in.}$$

(2) Necessary bearing value of one rivet on web. In order to determine the thickness of the web required in this case, it is necessary to know the number of rivets used in the end connection. We have already found that the depth of stringer required when a single row of rivets is used in the hitch-angle is 28 in. (page 70). As our stringer is only 24 in. deep, we will need to

use a wide legged connection angle in order to insert a sufficient number of rivets. With a hitch-angle having a 6-in. leg against the web and a distance between rows of rivets of 2-1/2 in., the minimum allowable rivet spacing is 1-3/4 in. in two lines. This detail may be made as shown on Plate I. It will be seen that there are seven rivets which pass through the hitch-angle, tight filler and web, and a maximum of four rivets through the tight filler and web only. This will give a maximum of eleven rivets

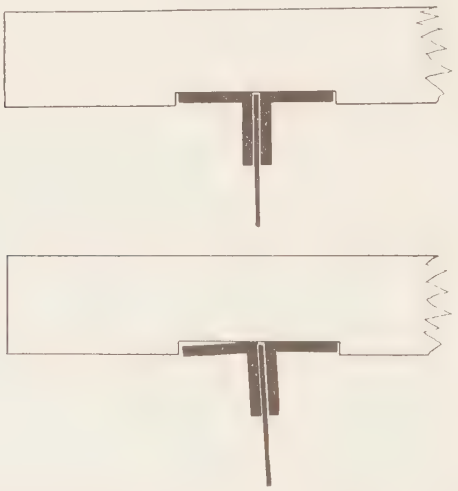


FIG. 51.—Effect on flange angles caused by deflection of tie (exaggerated).

in this connection. The necessary bearing value of one rivet then is $\frac{83100}{11} = 7582$ lb. The necessary thickness of web for this bearing value is $\frac{7582}{7/8 \times 24000} = 0.361$ in.

(3) Before proceeding to find the minimum rivet spacing for the flange rivets, it is necessary to know the size of the flange angles although the thickness need not be accurately computed at this point. So far we have tacitly assumed a 5-in. angle against the web. As a general proposition, flange angles with narrow horizontal legs are preferable for stringers provided they are made of sufficient width not to cut into the under side of the ties, and are also wide enough not to too materially reduce the allowable compressive fiber stress in the top flange. As the

rail is located to one side of the stringer and not directly over it, there is more or less deflection of the tie which results in the bending of the flange angle as illustrated on an exaggerated scale in Fig. 51. The wider the flange angle, the more severe this bending effect is; also, the wider the flange angle, the greater the allowable compressive fiber stress and the greater is the minimum allowable thickness of the angles. (Spec. ¶'s 31, 47.) The net result of these considerations is that for the *E* 60 loading which is now specified by so many roads, a 6×4 angle is used in the stringers with the 4-in. leg horizontal. For the *E* 50 loading, a 5 in.×3-1/2-in. angle will generally give satisfactory results. The 5 in.×3-1/2-in. is what we will assume in this case. The minimum rivet spacing which can be used in a 5-in. angle leg is determined by the same method we used previously in finding minimum pitch in the 6-in. hitch-angle leg. Referring to the table of gages on page 275 we find $g_2 = 1\text{-}3/4$ in. The minimum spacing then equals 2-1/2 in. (Table 35, page 274.) The longitudinal shear per unit of length between the web and the flange angles is found by dividing the maximum shear by the effective depth of the stringer. We do not know the effective depth as yet and so must assume it. A fair assumption is that it is 2 inches less than the depth of the web.

Longitudinal shear per inch of length is $\frac{83400}{22} = 3790$ lb.

The rivets in the top flange are evidently subjected to a vertical component of stress due to the fact that the ties rest on top of the flange angles. Therefore, the weight of the track together with any load which may be upon it is transmitted to the web of the stringer through the flange angles and the rivets connecting them to the web. This vertical component is computed as follows: A wheel load should be assumed as distributed over three ties (Spec. ¶ 9). As the ties we have designed are 7 in. in width and spaced 6 in. apart, we should have one wheel load or 25,000 lb. distributed over a length of $3\times7+2\times6=33$ in. This gives us a load per inch of $\frac{25000}{33} = 750$ lb. If we add 100 per cent. impact the vertical component per inch of length is $750\times2=1500$ lb.

$$3790^2 = 14{,}364{,}100$$

$$1500^2 = 2{,}250{,}000$$

Resultant $= 4075^2 = 16{,}614{,}100$ Approx. (Cambria handbook table of squares of numbers).

The resultant of 3790 and 1500 equals 4075 lb. This is the stress to be transferred from the flanges to the web per inch of length of stringer. As the rivets must be spaced at least 2-1/2 in. apart each rivet must carry at least $2.5 \times 4075 = 10{,}200$ lb. A 7/8-in. shop rivet will carry this stress easily in double shear and therefore it will be limited by bearing against the web. The web thickness must then be

$$\frac{10200}{7/8 \times 24000} = 0.486$$

(4) The subject of web stiffeners is a mooted one and there are many opinions and formulæ for determining their spacing. A discussion of this question has been given on page 43 et. seq. In this case we will apply the guides given in the specifications we are using (Spec. ¶ 81).

To avoid stiffeners the thickness of web must in any case not be less than 1/60 of the distance between flange angles or $\frac{14.25}{60} = 0.238$ in.

In the formula given $d = \frac{t}{40}(12{,}000 - s)$, d may be taken as the clear distance between flange angles or 14.25 in. s should be taken as the shear per square inch on the gross area of the web.

$$s = \frac{83400}{24t} = \frac{3475}{t}$$

Then $$14.25 = \frac{t}{40}\left(12{,}000 - \frac{3475}{t}\right)$$

$$14.25 = 300\,t - 87$$

$$300\,t = 101.25$$

$$t = 0.3375$$

The minimum thickness of web which can be used and avoid stiffeners is then 0.3375 in.

In our case, the minimum practicable spacing of flange rivets (Case 3) 0.486 limits the thickness of the web. We will make the web 1/2 in. thick. In any case, the web must not be less than 3/8 in. thick as stated in the specifications (¶ 40). The reason for this requirement in the specifications is that thinner metal will not so well withstand buckling and corrosion, particularly the latter.

Design of Flanges. The resisting moment of the stringer may be considered to be exerted by a couple each of whose forces acts along the center of gravity of the flanges and whose moment arm is called the effective depth. A certain part of the web may be considered as forming flange area (see page 39).

The effective depth must be assumed to begin with. A fair value to assume is two inches less than the depth of the web for this type of girder. The required flange areas then are to be tabulated as shown below.

$$\text{Approximate flange stress} = \frac{173750 \times 12}{22} = 95{,}000 \text{ lb.}$$

	Bottom flange *net*
Area required,	$\frac{95000}{16000} = 5.94$ sq. in.
Web equivalent 1/8×1/2×24,	1.50 sq. in.
	4.44 sq. in.
Two L's 5×3-1/2×3/8 (6.10-.75),	5.35 sq. in.

The tension flange should be designed first as it is necessary to reduce the working stress in the top or compression flange. This reduction is made to ensure the lateral stiffness of the compression flange of the stringer. It is generally made to vary inversely as the width of the flange and consequently the working fiber stress in compression cannot be determined accurately until the width of the flange is known. The working fiber stress in tension is known and consequently the sizes of the tension flange may be readily determined. The width of the compression flange is as a rule made the same as that of the tension flange. To explain further the conditions in the compression flange, it should be understood that the top or compression flange has a tendency to buckle. The web prevents its buckling vertically, but the flange has only its own stiffness to prevent it from buckling sideways. It acts then as a more or less restrained column. To guard against this buckling, the allowable fiber stress in the compression flange is limited by modifying its working stress by applying a formula similar in form to one type of column formula but with different constants. The reduction in fiber stress is made to depend upon the ratio of unsupported length to width of flange.

According to ¶ 31 of the Specifications, the allowable fiber stress in compression flanges is $16{,}000 - 200 \frac{l}{b}$. This value may

be roughly deduced from the column formula, $16{,}000-70\,\frac{L}{R}$ by considering that the horizontal portion of the compression flange is substantially a rectangular column. In a rectangular column the radius of gyration $R=\frac{b}{\sqrt{12}}=\frac{b}{3.46}$. Making this substitution for R in the column formula, we obtain $16{,}000-240\,\frac{L}{b}$. In settling upon the value of $200\,\frac{L}{b}$, the framers of the Specifications evidently intended to make some allowance for the bracing effect of the web in the horizontal direction. In large and important structures the top flanges of the stringers are often tied together by means of a system of braces similar to the lateral bracing on a truss or girder bridge. (Fig. 52.)

Fig. 52.—Type of bracing for top flanges of stringers, used in long panels.

This has the effect of securing the flange against buckling side-wise at the points of attachment of the bracing, and breaks it up into a series of short columns so far as horizontal buckling is concerned.

After having designed the tension flange, the effective depth should be computed accurately before designing the compression flange, as a change in the effective depth may change the whole design.

From the handbook we find the center of gravity of a 5 in.$\times$ 3-1, 2 in. $\times$3, 8 in. angle is 1.61 in. from the back of the shorter leg.

The actual effective depth is $24.25-2\times1.61=21.03$ in.

Actual flange stress $\frac{173750\times12}{21.03}=99{,}000$ lb.

See Spec. ¶ 20, 27, 34.

Top flange (gross)		Bottom flange (net)	
@ $16{,}000-\frac{200\times135}{7.5}=12{,}400$ lb. per sq. in.		@ 16,000 lb. per sq. in.	
$\frac{99{,}000}{12{,}400}=$	7.95	$\frac{99{,}000}{16{,}000}=6.18$ sq. in. required.	
Web equiv. 1/8$\times$24$\times$1/2	1.50		1.50
Two	6.45		4.68
5 in. $\times$3-1/2$\times$7/16 in. L's	7.06		6.20

This section is a little heavier than the one assumed and the effective depth is slightly less being 20.99 in. instead of 21.03 in. This would have the effect of slightly increasing the flange stress and consequently slightly increasing the required area. The two 5 in.×3-1/2 in.×7/16 in. angles are enough larger than the required area, however, to more than compensate for the reduced effective depth. It should be stated here that it is a bad plan to "skin" sections on a railroad bridge as there is always the possibility of future increase in loads requiring larger sections. Consequently it is always better to be a little on the safe side, although an arbitrary increase in section should not in general be made for this purpose.

The top and bottom flanges of girders should generally be made of the same size and thickness in order that end hitch angles and intermediate stiffeners may be fitted over the flanges with a minimum of trouble. Some specifications require this.

Flange Rivets.—The next step in order is to compute the spacing of flange rivets. We have already computed the necessary thickness of web for the minimum pitch of flange rivets. As, however, we did not use the exact thickness of web computed (0.485), but 1/2 in. (0.50), it is necessary to now compute accurately the spacing of the flange rivets. The rivets in the bottom flange carry horizontal components of stress only. The rivets in the top flange must not only transfer the shear from the web to the flanges but must also, as explained before, transfer the vertical loads from the wheels to the web through the flanges. Generally the rivet spacing in the top flange should be computed and the same rivet spacing adhered to in the bottom flange. This enables the top and bottom angles to be punched from the same template which is of great importance both in lessening work in the template shop and in simplifying the assembling in the riveting shop. In explanation of the last statement, if the flange angles differed slightly in thickness or in their number of rivets, the men would be likely to pick up the wrong angle and after trying it in place discard it for the right one, all of which would waste time and therefore money; further, both ends of the angle should have the same rivet spacing if possible in order that they may fit without being turned around after trying in place.

The horizontal component of the stress on one lineal inch is found by dividing the shear at the chosen point by the effective depth of the stringer at that point. This is what is known as the

"approximate method" and gives results slightly on the safe side when applied to girders having flanges similar to the ones we are dealing with in this problem, that is, like Fig. 59, *a*, *b*, and *c*. When other forms of flange, such as those shown in Fig. 59, *d*, *e*, *f*, and *g* are used, the only safe formula to use is $S = \frac{VQ}{I}$ where

S = horizontal shear per linear inch of girder.
V = total vertical shear at section under consideration.
Q = statical moment about the neutral axis of the girder of part attached to girder by the rivets under consideration.
I = moment of inertia of girder about its neutral axis at the section under consideration.

For a proof of this formula the reader is referred to any standard work on structures or mechanics of materials. The vertical component of stress on one linear inch of stringer has already been found and explained. (See page 75.)

The horizontal component should now be computed using the exact effective depth instead of the approximate heretofore used. This will now be done without further explanation.

$V.\ C. = 1500$ lb. per lineal inch.

$H.\ C. = \frac{83400}{20.99} = 3980$ $\qquad 1500^2 = 2{,}250{,}000$

$3980^2 = 15{,}840{,}400$

$4250^2 = 18{,}090{,}400$

Resultant = 4250 lb.

The value of one 7/8-in. rivet in bearing on the 1/2-in. web plate is $24{,}000 \times 7/8 \times 1/2 = 10{,}500$ lb. In double shear @ 12,000 lb. $= 2 \times 0.6 \times 12{,}000 = 14{,}400$ lb. The required rivet spacing then is $\frac{10500}{4250} = 2.48$ in., say 2-1/2 in. The rivet spacing at the quarter-point is determined in a similar manner.

Horizontal component $= \frac{52850}{20.99} = 2510$ $\qquad 2510^2 = 6{,}300{,}100$

Vertical component $= 1500$ $\qquad 1500^2 = 2{,}250{,}000$

Resultant = 2930 lb. $\qquad 2930^2 = 8{,}550{,}100$

Spacing is $\frac{10500}{2930} = 3.58$ or 3-1/2 in.

The rivet spacing should never be made more than the computed distance except in certain cases around stiffeners and end connections where, owing to the presence of other pieces, it is

impossible to use the computed spacing. In such cases, the spacing should be reduced below the computed amount at the nearest practicable point in order to make the average rivet spacing conform to the computed spacing. It is unnecessary to figure the rivet pitch at any other points as it would not be changed at more than one point in the half-length of the stringer.

Connection of Stringer to Floor Beam.—The final design for the end connection to the floor-beams should now be made. There is no method of figuring hitch angles which will give sizes as thick as are considered good design. The judgment of the designer must be relied upon to fix these sizes. The angles connecting stringers to floor-beams in railroad bridges should not be less than 3-1/2 in. $\times$3-1/2 in.$\times$9/16 in. The 3-1/2 in. leg is specified in order to get a size of angle in which the 7/8-in. rivet may be readily used. The 9/16-in. thickness is specified because the ends of the stringers are, in the best practice, milled or planed so that they are to exact length. It is necessary to provide sufficient thickness of metal in the hitch angles so that it will not be reduced to too small an amount by this milling. In this connection it should be noted that ¶'s 19, 43 and 81 in the Specifications do not apply and ¶'s 60, 70 and 145 do apply. Referring to what has been said before about this connection (see page 69), it is seen that a tight filler must be used. The total number of rivets used to fasten the angles and tight fillers to the web will be determined by dividing the maximum end shear on a stringer by the value of one rivet in bearing on the web or double shear, whichever is the lesser value. In this case, bearing on the web is the lesser. No. of rivets $= \frac{83400}{10500} = 8$ rivets. The number of rivets that must pass through both the hitch angles and tight filler is determined by dividing the end shear by the value of one rivet in bearing on the aggregate thickness of the web and the tight fillers (1-3/8 in. in this case) or in double shear. In this case double shear is the lesser. Number of rivets equals $\frac{83400}{14400} = 6$. Then six of the rivets must pass through hitch angles, tight filler and web, and the remaining two need pass through the tight filler and web only. Generally rivets are spaced opposite each other, which usually leads to more rivets being used through the tight filler and web only than are necessary. This is a place where rivets are likely to become loose in the course of time. Therefore, a few extra rivets may reduce

the cost of maintenance of the bridge. The final arrangement of this detail is shown on Plate I.

The dead weight of the stringer must now be computed accurately in order to see whether any revision of the design of the stringer is necessary, and also to enable the designer to use the actual weight in his further computations. Too much emphasis cannot be placed upon the importance of obtaining and using a correct value for the dead load of each part at the earliest possible point in the computations.

One 24 in. ×1/2 in. web 11.25 ft. @ 40.8 lb. per foot	458
Four 5 in. ×3-1/2 in. × 7/16 in. angles 11.25 ft. long @ 12 lb. per foot	540
Four 6 in. ×4 in. ×9/16 in. hitch angles 1 ft. 11-3/8 in. long @ 18.1 lb. per foot	141
Four 8-3/4 in. ×7/16 in. fillers 1 ft. 2 in. long @ 13.02 lb. per foot	61
200-7/8-in. rivet heads @ 24.29 lb. per 100	50
Total weight of one stringer	1250

The assumed weight of the stringer was 100 lb. per foot or 1125 lb. The actual dead weight is then more by 125 lb. than that assumed and the actual dead moment is also slightly more than assumed. A correction of 63 lb. in a total shear of 83,400 is evidently too small to make recomputation necessary. The assumed dead shear in the summary, 1700 lb. is slightly smaller than the amount, 1750 lb., that exact computation gives. We can, however, say that the assumed practically equals the actual. It is generally unnecessary to make any correction unless the error is more than one per cent. of the assumed weight of the structure for spans of the ordinary types and lengths. We will make the necessary correction in the moments in the summary although it will be unnecessary to revise the shear or the design of the stringer. For certain types of bridges, such as the cantilever, it may be desirable to reduce the error to an amount less than this, and on very large bridges, the dead weight of the paint is an item which should not be disregarded.

A summary of the principal dimensions of the stringer should be made at this point so that the different items needed in making the drawing may be readily found without hunting through several sheets of computations. This summary is as follows:

Stringer web 24 in. ×1/2 in.

Each flange two L's 5 in. ×3-1/2 in. ×7/16 in.

Computed flange rivet pitch at end, 2.48 in.
Computed flange rivet pitch at quarter-point, 3.58 in.
Number of rivets connecting hitch angle to stringer, 6.
Total number of rivets connecting hitch angle and tight filler to stringer, 8.
Distance back to back of flange angles, 2 ft. 1/4 in.

FLOOR BEAMS

Weight of Floor Beam and Connection of Stringer. The design of the floor-beam, so far as its section is concerned, is generally similar to that of the stringer, although differing in important details. Its dead weight should be assumed to be about 225 lb. per foot and the total dead moment and shear upon it computed from this assumption and the known weight of the stringer. The number of rivets which can be put through the angles connecting the stringer to the floor-beam is limited unless the leg of the hitch angle in contact with the web of the floor-beam is made at least 5 in. wide. We will try not to use a wide legged angle here. By reference to the detail of this connection (see Plate I) and to the table on page 275, it will be seen that only seven field rivets can be put in each hitch angle. Fourteen field rivets must then be capable of carrying in single shear the maximum shear on one stringer or of carrying, in bearing on the web, the maximum panel concentration obtained on the floor-beam from the stringers. The first condition should be investigated by the student at this point. In the case we are considering the value of one 7/8-in. field rivet in single shear is 6000 lb. We will then require $\frac{83400}{6000} = 14$ rivets in this connection. The second condition cannot be determined until the thickness of the floor-beam web has been settled. Some engineers claim that more rivets than computation shows to be necessary should be put in this connection because the effect of these rivets is to make the stringers continuous beams; consequently some of the rivets are in tension and so the total number should be increased. Those who take an extreme view advocate using twice the computed number of rivets in this connection, basing their argument on the desirability of having no rivets in tension counted in shear and therefore counting only those rivets which are below the neutral axis as being effective to take care of the reaction of the stringers.

Floor Beam Web. The thickness of floor-beam web to be used is, in designs of this type, often determined by the minimum rivet pitch that can be used in the flange angles. The depth is usually determined by considerations of clearance as heretofore stated. In our case we will assume that the stringer is riveted to the web of the floor-beam. It is desirable to make the floor-beam as deep as practicable in order to make the flange angles as light as possible. By reference to Plate I, it will be seen that the top flange of the floor-beam may be put at a considerable distance above the top flange of the stringer without interfering with the rails. As a 9-in. depth of tie is used which is to be notched 1 in. over the top flange of the stringer, the floor-beam may extend upward a distance of 8 in. from the top of the stringer before striking the rail. Some clearance must be left between the rail and the top of the floor-beam. This will be sufficiently cared for if we extend the web of the floor-beam 6 or 7 in. above the top of the stringer. Wherever the clearances, depths, etc., make it practicable, it simplifies the details considerably to rivet the hitch angles of the stringer directly against the web of the floor-beam. This is done by raising the bottom flange of the stringer completely above the bottom flange of the floor-beam and making the clear distance between the flange angles of the floor-beam such that the stringer may be inserted without any difficulty. Generally 1 in. clearance, top and bottom, is sufficient. This would lead us to extend the floor-beam, assuming that we will use 4 in. $\times$4 in. flange angles in it, 5 in. below the stringer. It is possible of course to arrange this connection as shown in Fig. 50. This type of connection may be resorted to to obtain a greater depth of stringer than is otherwise possible, or, on the other hand, to make the floor-beam as shallow as possible. The added economy which might be obtained by using the deeper stringer would be partially, if not entirely, offset by the necessity of using tight fillers against the web of the floor-beam of a thickness equal to the floor-beam flange angles. These fillers are, of course, unnecessary where the stringers are riveted directly against the web. It is possible, in the case we have in hand, to extend the floor-beam 7 in. above the top of the stringer. This with the 5 in. below, makes a total depth of floor-beam of 36 in. The reader should understand that innumerable combinations are possible in plate girder designs, and that the one should be chosen which gives the best and most

economical design on the whole for the case in hand. No absolute set of rules can be set down which will cover every point which arises in the course of a design. We will make the web 36 in. deep in this case.

Design of Flanges.—The simplest way to proceed is to determine the thickness of floor-beam web required for shear, and then to design the flanges tentatively; revising the design if the minimum rivet pitch that can be used in the flange angles makes changes in the web necessary. This minimum pitch cannot be determined until it is known whether the flange angles will be of a size that will admit of one or two rows of flange rivets.

Thickness of web required for shear

$$\frac{112175}{36\times 10000}=0.311 \text{ or } 3/8 \text{ in.}$$

The design of the flanges is similar in its steps to the design of any plate girder when its depth is known and is as follows:

Approx. flange stress $\dfrac{420650\times 12}{34}=148{,}500$ lb.

Bottom flange area $\dfrac{148500}{16000}=9.27$ sq. in.

Web equiv. $1/8\times 36\times 3/8=1.69$

7.58 net

Two 4 in. $\times$ 4 in. $\times 5/8$ in. angles $=8.00$ net.

Assuming that 4 in. $\times 4$ in. $\times 5/8$ in. angles will do for both flanges, the next step is to find the true effective depth.

The true effective depth is $36.25-(2\times 1.23)=33.79$.

Actual flange stress $=\dfrac{420650\times 12}{33.79}=149{,}400$ lb.

The top flange may be considered to be braced laterally where the stringer is connected to the floor-beam. Its greatest unbraced length then is 6 ft. 6 in. and the allowable fiber stress in compression is

$$16{,}000-200\,\frac{78}{8.37}=14{,}150 \text{ lb.}$$

	Top (gross)	Bottom (net)
Required flange area	$\dfrac{149400}{14150}=10.56$	$\dfrac{149400}{16000}=9.34$

Web equivalent	1.69	1.69
	8.87	7.65
Two 4 in.×4 in.×5/8 in.	9.22	8.00

Pitch of Flange Rivets.—The next thing to determine is the rivet pitch in the flanges. This may be arrived at by either of the methods given heretofore, or by the following method which is better in this particular case. Find the stress in the flange angles at the center line of the stringers, and put enough rivets through the flanges and web between this point and the girder to take this stress from the flange angles into the web. This method is applicable in all cases over a distance where the shear is constant. The application of this method to this particular case is as follows: The rivet pitch is to be made the same in both flanges, as it will facilitate and cheapen the shop work to have the flange angles on the floor-beams all exactly alike. The largest stress in the flange angles will then have to be determined. This may be done by multiplying the total flange stress by the ratio of the area of the angles and cover plates, if any, to that of the whole flange. The whole flange consists of the angles, cover plates, if any, and web equivalent.

The stress in the flange angles at the center line of the stringer is so nearly equal to that at the center of the floor-beam, that the latter value may be used. It is found as follows:

For compression flange	For tension flange
$\frac{9.22}{10.91}\times 149{,}400 = 126{,}250$	$\frac{8.00}{9.69}\times 149{,}400 = 123{,}350$
$10.91 = 9.22 + 1.69$	$9.69 = 8.00 + 1.69$

The larger of these (126,250 lb.) is the one to use. This method provides for all the stress that the flange does bear under the assumed loadings. It is better practice to "develop" the flange, that is, to provide enough rivets to take the whole of the stress that the flange angles and cover plate, if one is used, can bear, assuming that they are stressed up to the maximum allowable value in tension and compression. The stress to be provided for when figured in this way will be

Top flange	Bottom flange
$9.22\times 14{,}150 = 130{,}460$ lb.	$8.0\times 16{,}000 = 128{,}000$ lb.

The larger of these, 130,460 is the one to use and we will compute the required thickness of web using this stress rather than the 126,250 lb. previously found.

Referring to Plate I, it will be seen that the first rivet cannot be closer to the center of the girder than 5-1/4 in. plus one-half the thickness of the girder web, plus the thickness of the girder flange angles. The distance 5-1/4 in. comes from assuming a 3-1/2 in. hitch angle in the connection of the floor-beam to the girder, 1/4 in. clearance between the hitch angle and the end of the flange angle, and 1-1/2 in. from the end of the flange angle to the first flange rivet. As the girder is not yet designed, these values must be assumed. If we assume a 3/8-in. web and a 9/16-in. flange angle, we must add to 5-1/4 in., 3/4 in., making 6-1/4 in. from the center line of the girder to the first flange rivet on the floor-beam. This leaves 3 ft. 3 in. for the rivets. It is possible to put 14 rivets in this distance. This would require the value of each rivet to be $130{,}460 \div 14$, or 9318 lb. per rivet. To obtain this value for one rivet in bearing at 24,000 lb. per square inch requires a web thickness of $9318 \div (24{,}000 \times 7/8) = 0.4437$, or practically 1/2 in. As this calls for an increased thickness of web (1/2 in.) where we had assumed 3/8 in. we will revise our design using a 1/2-in. web, and see whether we can reduce the thickness of the flange angles.

	Top flange	Bottom flange
Required area	10.56	9.34
Web equiv. 1/8×1/2×36	2.25	2.25
	8.31	7.09
Two 4×4×5/8 angles	9.22	8.00

The design will then remain as before using a 1/2-in. web and two 4×4×5/8-in. flange angles.

The pitch of flange rivets between the stringers would be determined from the shear, which is almost zero. The student should determine what this pitch should be. It is not considered good practice to use a rivet pitch in any case which is greater than 16 times the thickness of the thinnest part through which the rivet passes nor more than 6 in. Greater pitches than this do not hold the parts together as well and a firmly as they should be held. Fig. 53 shows how plates in compression may separate if not riveted together at sufficiently frequent intervals.

Fig. 53.—Buckling of plates allowed by rivets being too far apart.

General.—One point that should be borne in mind in designing floor-beams for open-floor railroad bridges is that the top flange should not be any wider than is absolutely necessary on account of causing too great a space between adjacent ties. It is, of course, possible to put a wooden block on top of the floor-beam under the rails but there are some objections to this. It causes a hard spot in the track owing to the fact that while, due to the relative position of the rails and stringers, the ties spring more or less under the live load, the rail where it is supported directly on the floor-beam cannot deflect so much. This causes a high or hard spot in the track. On the other hand, separating adjacent ties too much causes a soft spot, owing to the deflection of the rail under the loads.

Stringer Connection.—The details of the connection of the stringer to the floor-beam should now be finally settled. Enough rivets should be provided to take care of the maximum panel concentration, the rivets being limited by bearing on the $\frac{1}{2}$-in. web of the floor beam.

The maximum panel concentration brought to the floor-beam from the stringers equals

Weight of track 200×11.25,	2,250
Weight of one stringer,	1,250
Live load	55,500
Impact $\frac{300}{300+22.5} = 93$ per cent.,	51,600
Total,	110,600

$$\frac{110600}{8750} = 13 \text{ field rivets.}$$

By reference to Plate I, it will be seen that 14 rivets can readily be obtained in this connection. In any case 14 are required in single shear to transfer the maximum shear on one stringer to the floor beam.

Connection of Floor-beam to Girder.—There is usually no difficulty in securing a satisfactory end-connection between the floor-beams and girders, especially with the type of floor-beam shown on plate I. The design of the number of rivets required in the hitch-angles is so simple that the student should find no difficulty with it. He is expected to make this computation at this point in his work with no further instruction.

Construction at End of Floor-beam.—It is necessary to brace the top flange of the girder wherever conveniently possible, in

order to secure it against buckling sideways. The most convenient points in this type of bridge are at the floor-beams. A triangular plate, called a gusset, is put between the girder and the top of the floor-beam. This detail is quite often arranged as shown in Fig. 54.

It is generally considered better to use a detail of the form shown on Plate I for this gusset, and this form has certain advantages which will appear as we proceed with the design. Its disadvantage is the difficulty of securing a splice of sufficient strength where the gusset and the web are joined together. As will be seen from the figure the end portion of the web is separate and nearly as high as the girder. The whole web is composed of three pieces shaped as shown in Fig. 55.

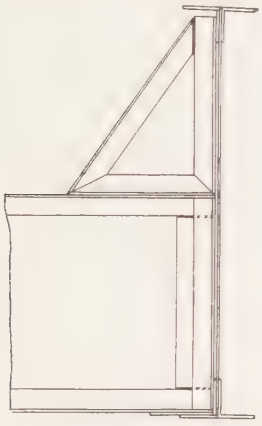


FIG. 54.—A type of floor-beam connection and gusset.

It would evidently be impracticable

FIG. 55.

to make the web in one piece with a re-entrant angle as shown at *b* in figure 55.

Design of Web Splice.—The design of the splice will now be taken up. The element which will fix the maximum distance to which the gusset can be extended is the clearance diagram. (Spec. ¶ 6) (Fig. 56). This clearance diagram is not a universally accepted standard and varies on different railroads. Fig. 57 page 91 shows the more usual form of clearance diagram and the dimensions as used by various roads. In general, the angle which the edge of the gusset makes with the horizontal need not be less than 45 degrees. Generally if it can be made to extend halfway from the girder to the stringer without infringing on the clearance it will be sufficient.

Assuming that it is so located, we will now design this splice. It is evidently subjected to both bending and shearing stresses. The bending moment to which the floor-beam will be subjected

at this point, half way between the girder and stringer, will evidently be almost exactly one-half of the maximum moment on the floor-beam. As the floor-beam can only receive load through the stringers, this proportion of the full amount can never be exceeded. Hence if the web be spliced for the maximum shear which it receives, and for one-half of the moment which it can carry, the splice will be as strong, considering the work it is called on to perform, as any other portion of the beam. This will not be true if there are cover plates on the floor-beam, and in the latter case the web may carry its full stress at the splice. Where there are cover

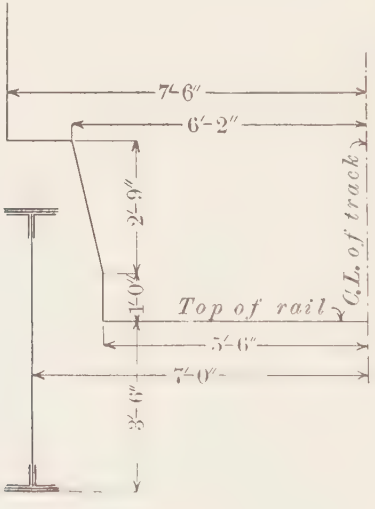


Fig. 56.

plates the type of detail shown in Fig. 54 will generally be necessary because of the difficulty of securing an adequate web-splice for full shear and moment. As many rivets as it is practicable to put in will be needed in this splice. We will, therefore, assume two rows on each side of the seam at substantially a 3-in. spacing. In order to avoid unnecessary recomputation the rivet spacing should be laid out before trying to compute the splice. We will assume the splice shown on Plate I. Where a splice is so shallow in proportion to its width, the stress on the remotest rivet should be computed about the center of the group of rivets on one side of the seam, and not about the intersection of the neutral axis of the beam and each successive row of rivets. The latter method

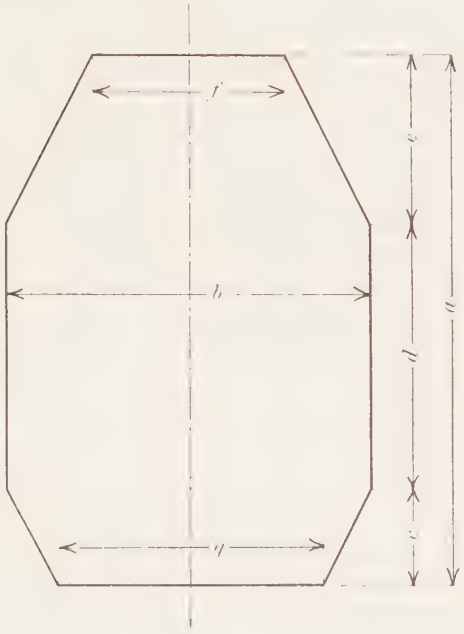


FIG. 57.

Road	Dimensions a	b	c	d	e	f	g
New York New Haven & Hartford in Canada.	22′–0″	16′–0″	3′–0″	15′–0″	4′–0″	8′–0″	13′–0″
New York Central	22′–0″	15′–0″	4′–0″	11′–0″	7′–0″	8′–0″	11′–0″
Lehigh Valley	22′–0″	14′–0″	6′–6″	11′–3″	4′–3″	11′–0″	10′–6″
Illinois Central	23′–0″	16′–0″	4′–0″	15′–0″	4′–0″	8′–0″	11″–0″
Chicago and Alton.	24′–0″	15′–0″	4′–0″	15′–0″	5′–0″	6′–0″	11′–0″
American Ry. Eng. Assoc.	22′–0″	14′–0″	4′–0″	14′–0″	4′–0″	6′–0″	10′–6″

gives sufficiently close results when the depth is great in proportion to the width of the splice.

c in Fig. 58 is the center about which the moment of the group of rivets shown, which comprise one-fourth of the total number in the splice, will be computed. The rivet marked a is evidently the one most stressed. If P be the stress on rivet a due to torsion on the joint, $P \times d$ will be its moment about c. On any other rivet as b at a distance d' from the center the force acting will be $P' = \frac{P}{d} \times d'$ and its moment about the center c will be $\frac{P}{d} \times d'^2$. The moment for the whole group will equal the $\frac{P}{d} \times \Sigma d'^2$. The finding of all of the distances d' will evidently be somewhat laborious if each one is found separately. In any case d will have to be found which may readily be done by using a table of squares.

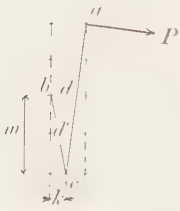


Fig. 58.

$(1\text{-}1/2 \text{ in.})^2 = 0.015625$
$(12\text{-}3/8 \text{ in.})^2 = 1.063480$
$(12\text{-}7/16 \text{ in.})^2 = 1.079105$

Now note that $(d')^2$, which is what is wanted for use in the summation, equals $m^2 + k^2$. Note also that k^2 is a constant for each row of rivets, and will occur once in the summation for each rivet. Then we may write for the two rows of rivets:

$3.375^2 =$	11.4
$6.375^2 =$	40.6
$9.375^2 =$	88.0
$12.375^2 =$	153.0
	293.0 for one row.
	2
	586.0 for two rows.

$2\times1.5^2\times4=$	18	k^2 for the eight rivets in two rows
	604.0	$\Sigma d'^2$ for all rivets in one-half of one side of whole splice except those on neutral axis.
	2	
	1208.0	$\Sigma d'^2$ for all rivets in one side of splice.
$2\times1.5^2=$	4.5	k^2 for two rivets on neutral axis.
	1212.5	

Then $\frac{P\times1212.5}{12.44}$ = the torsion the rivets in the splice carry. The moment which they may be obliged to carry is one-half of that which the web might be called upon to bear at the center of the floor-beam. $1/8\times1/2\times36$ is the web equivalent or the part of the web that may be counted as flange area. 33.79 is the effective depth and 16,000 is the allowable stress on the flange in pounds per quare inch. From this we obtain

$$1/2\times1/8\times1/2\times36\times33.79\times16{,}000=608{,}200 \text{ in.-lb.}$$

$$P=\frac{608200\times12.44}{1212.5}=6240 \text{ lb.}$$

The vertical shear which one rivet must carry is 111,975 lb. divided by 18 rivets, or 6221 lb. on each rivet. The most stressed rivet is shown in Fig. 58; the computation of the resultant stress is as follows:

$$\text{Total } V.C.\ 6221+\frac{1.5}{12.44}\times6240=6973 \text{ lb.}$$

$$\text{Total } H.C.\ 6240\times\frac{12.375}{12.44}=6206 \text{ lb.}$$

$$6970^2=48{,}580{,}900$$
$$6210^2=38{,}564{,}100$$
$$9335^2=87{,}145{,}000$$

The resultant stress on the remotest rivet is 9335 lb. As the value of a rivet in this case is limited by bearing on the 1/2-in. web or 10,500 lb. the splice as designed is of sufficient strength. It is not worth while to try to rearrange the spacing or to reduce the number of rivets in the splice in order to bring the unit stresses on the rivets up to the maximum allowable amount.

In any case it would not be possible to save more than one or two rivets. This would not be good practice in a splice of this kind.

The splice plates will obviously be of sufficient strength if their section modulus computed by considering the gross area is made the same as that of the web.

$$1/6\times1/2\times36\times36=1/6\times2t\times28\times28$$

The thickness of each splice plate, $t=0.414$ or 7/16 in.

Cutting off Flange Angles.—The next step is to find whether the flange angles of the floor-beam can be cut off near the girder in order to use the type of detail of end connection shown in the lower right-hand corner of Plate I. In order to cut off these flange angles the web must be capable of carrying the whole moment on the floor beam at the last rivet connecting the flanges to the web. This rivet is located 6-5/16 in. from the center line of the web of the girder. This dimension is determined from the finished drawing which would not be available at this point. In ordinary cases it is sufficient to assume this distance as 6-1/2 or 7 in. The bending moment at this point, neglecting the negative moment due to the weight of the part of the floor-beam between this point and the girder, is equal to the maximum end shear on the floor-beam multiplied by the distance from this rivet to the center line of the girder. As the height of the web varies somewhat and generally cannot be accurately determined until the drawing is made, the best way to proceed is to find out how deep the web needs to be to carry this moment. This may be done by using the formula for rectangular beams. It assumes that no rivet holes are subtracted, in other words, that the gross section may be used.

$$1/6\times16{,}000\times1/2\times h^2=112{,}175\times6.3125$$
$$h=22.7 \text{ in.}$$

As this is less than the depth of the floor-beam, the web is amply sufficient to take care of the moment it will be called upon to bear if the flange angles are cut off as shown on Plate I.

Weight of Floor-beam.—The weight of the floor-beam must be computed next. In order to obtain it with reasonable accuracy, the depth of the girder must be known. (See page 99.) In figuring these weights the exact length determined from the finished drawing has been used. It is not practicable for the student to make his computations as closely as is done below at

this stage in his design, but he should be able to determine the lengths of the different parts within an inch or two of the correct value.

One 36-in. $\times 1/2$-in. web plate 10 ft. 3 in. long @ 61.2 lb.,	= 627
Two webs 1/2 in. thick, 1141 sq. in. in area, $\frac{1141}{1728} \times 1/2 \times 480 \times 2$	= 317
Four splice plates 12 in. $\times 7/16$ in. $\times 2$ ft. 4 in.,	= 167
Four 4-in. $\times 4$-in. $\times 5/8$-in. *Ls* 13 ft. 3/8 in. long,	= 830
Four 3-1/2$\times$3-1/2$\times$1/2-in. hitch *Ls* 5 ft. 6 in. long,	= 244
Four 3-1/2$\times$3-1/2$\times$3/8-in. *Ls* 2 ft. 10 in. long,	= 97
Two hundred 7/8-in. rivet heads @ 24.29 lb. per 100,	= 50
	2332

This may be called 2340 lb. It should be noted that the weight of the stringer (see page 82) is 1250 lb. actual which agrees fairly well with 1125 lb. assumed. The weight of the floor-beam is 2340 lb. actual compared with 3150 lb. assumed.

Actual Dead Stresses.—The dead stresses so far as we have gone are less, by a considerable percentage, than those assumed. The weight of the lateral bracing is to be included in the weight of the floor system, which will raise the dead load on the girder at the panel points somewhat. On this account it will probably be as well to let the assumed dead moments and shears conveyed to the girder from the floor system stand as they are until the design of the girder is completed. It will be well, however, to revise the dead moments and shears on the floor-beams at this point in order to see whether the reduction of dead load from that assumed will affect its design.

The true dead stresses on the floor-beam are as follows:
The maximum shear equals

Weight of one stringer and 1/2 of one panel length of track,	3500 lb.
1/2 wt. of one floor-beam,	1170 lb.
Total,	4670 lb.

The maximum dead moment at center of one floor-beam equals

from stringer and track 3500×3.75	= 13,100 ft.-lb.
from floor-beam $1/8 \times 2340 \times 14$	= 4,100 ft.-lb.
Total,	17,200 ft.-lb.

The true moment on the floor-beam then is

Live,	208,000 ft.-lb.
Impact,	194,000 ft.-lb.
Dead,	17,200 ft.-lb.
Total,	419,200 ft.-lb.

compared with 420,650 assumed.

The required flange area, obtained by multiplying the area as heretofore determined by the ratio of actual to assumed moment is:

Top Flange	Bottom Flange
$\frac{419200}{420650} \times 10.72 = 10.69$	$\frac{419200}{420650} \times 9.50 = 9.48$

By referring to the computations on page 87 it will be seen that no reduction can be made in the floor-beam section. In cases where the stringer rests on the bottom flange of the floor-beam, it will be necessary to use a filler under the end of the stringer (see Fig. 50), and this filler will need to be made tight, as it will generally be impracticable to increase the number of rivets required by 50 per cent. and use a loose filler. The number of rivets required in the tight filler outside the hitch angle cannot be computed, as the filler would not be required if the bottom flange of the stringer did not come against the bottom flange angle of the floor-beam.

Summary.—The summary of the principal dimensions of the floor-beams is as follows:

Web, 36 in. $\times 1/2$ in.

Each flange 2 L's 4 in. $\times 4$ in. $\times 5/8$ in.

Number of flange rivets between center line of stringer and end of flange angles, 14.

Flange rivet pitch between stringers, 6 in.

Number of field rivets connecting stringer to floor-beam, 14.

Web splice plates, two 12 in. $\times 7/16$ in.

Rivets in splice plate, two rows spaced from the center outward as follows: First space, 3-3/8 in., then three spaces at 3 in.

Required number of shop rivets to connect floor-beam to hitch angle

$$\frac{111770}{10500} = 11 \text{ rivets.}$$

Required number of field rivets to connect hitch angle to girder

$$\frac{111770}{6000} = 19 \text{ rivets.}$$

THE GIRDER

General.—The live and dead moments and shears on the girder, except those caused by its own weight, have now been computed. The next step is to determine the depth of the girder. Provided there are no external limiting circumstances, such, for instance, as interfering with the clearance diagram, the girder may be made of any depth the designer chooses. It is evident that a shallow girder, on account of its heavier flanges, is heavier and consequently more costly than a deep one would be. One great advantage of plate girders as compared with trusses for the same span and loading is their stiffness. As a general proposition, the deeper the girder is, the stiffer it is and consequently it is desirable to use as great a depth as practical limitations of size of material, clearances, etc. will allow, in order to obtain the stiffest possible construction.

Economic Depth.—There are many formulas given for determining the depth of plate girders, most of which claim to give the "economic depth." They may do so in certain cases but the author is not willing to recommend any of them. In a girder composed of flange angles and web alone, with no web stiffeners, the economical depth will be that depth at which the minimum size of flange angle is just sufficient in area to carry, in comabination with the web, the maximum bending moment. This minimum size of flange angle is determined by various elements. The element which will probably fix the size of the outstanding legs of the compression flange angles within comparatively narrow limits is the reduction in fiber stress required to offset the buckling tendency of the compression flange. This reduction may become very large when very narrow flanges are used on long girders. The use of cover-plates, which are needed through only a portion of the length of the girder, is economical because the resulting reduction in height and therefore weight of web extending through the whole length of the girder is greater than the weight added by the cover-plates, which extend through only a part of the

length. The reduction in weight of web stiffeners, which is accomplished by reducing the height, also means a saving.

There are also other considerations: such as the necessity of using stiffeners in deep webs, and the difficulty of handling light and deep girders in the shop, which set a limit above which it is undesirable to go. Some investigations which the author has made show that the economic depth depends almost wholly on the web stiffener formula used. For an *E*60 loading using either the American Railway Engineering Association or the New York, New Haven and Hartford Railroad specifications, the economic depth is close to one-eighth of the span. In our case this would give a depth of 67-1/2 in. The maximum depth which can be used is fixed by limitations of shipment, and also by the width of plates which can readily be obtained. From 10 ft. to 11 ft. deep is the present maximum and limitations of shipment are such that it is unlikely that this depth can ever be materially exceeded. The clearance diagram (Spec. ¶ 6, Figs. 56 and 57), may also set a limit upon the depth, especially in the case of multiple track bridges.

For an illustration of this see Fig. 56.

The distance from top of rail to bottom of girder will be in our case, assuming that the floor-beam rests on the bottom flange of the girder, 24-1/4 in. (depth of stringer) +8 in. (depth of tie allowing 1 in. for notch) +5-3/4 in. (height of rail) +5/8 in. (thickness of floor-beam flange angles) + thickness of girder flange angles and the maximum thickness of cover plates—or say about 40 or 41 inches in all. Assuming a width of cover plates of 14 in. it is evident that the top of the rivets in the outer cover plate may come 3 ft. 11 in. above the top of the rail. This will give a maximum possible total depth of 7 ft. 5 in. over all. Allowing for cover plates, etc., this will give us a possible maximum depth of from 80 in. to 84 in. for the web. It is usual to use web plates of an even integral number of inches in depth, as these seem to be more readily obtained. The form of clearance diagram used in the N. Y., N. H. & H. specifications and shown in Fig. 56, is a little unusual. The more common form is similar to the one shown for Canada in ¶ 6 of the specifications. A composite diagram made up from the specifications of several roads is shown in Fig. 57. It is evident that in the case of a single-track bridge such as we have, when using the more usual form of clearance diagram, the depth of the girder can be increased by moving the girders out and lengthening

the floor-beams. This would materially increase the weight of the floor-beams, perhaps enough so to offset the possible gain from using a deeper girder. For bridges with several tracks, the girders must be located half way between tracks. They must not encroach on the clearance diagram and consequently the greatest possible depth is often quite small. Multiple-track roads having narrow track centers are, as a necessary consequence, obliged to build shallower and consequently more expensive through girders than are roads having wide centers. The shorter floor-beams which are possible with narrow centers partially offset this disadvantage.

It happens in our case that the maximum depth taking account of clearance is much greater than that obtained by using one-eighth of the span. In such a case several rough designs would be made varying the heights of web about 4 in. and computing simply web and flange sections and stiffener spacing. The weight of these different designs would be computed and the lightest one used. A series of such girders can be very rapidly designed by using the tables at the back of this volume. If desired a curve may be plotted using depths for the ordinates and weights for the abscissas. This curve would very closely indicate the economic depth for the case in hand. We will use a depth of one-eighth of the span, which gives a web depth of 68 in., in our design, unless some reason is found for altering this as we proceed.

Dead Weight and Dead Stresses.—The dead weight of the girders may be assumed at about 300 lb. per foot each; this figure includes stiffeners and other details.

The dead shears are readily found as follows: The largest dead shear due to the weight of the girder occurs at its end and equals $300 \times 22.5 = 6750$ lb. The dead shear due to the girder in panel B equals $300 \times 11.25 = 3375$ lb. The dead shear from the floor system in panel A equals $3/2 \times 4700$, or 7050 lb., and in panel B equals $1/2 \times 4700$ or 2350 lb. The total dead shear in panel A then equals $6750 + 7050 = 13{,}800$ lb. and in panel B equals $3375 + 2350 = 5725$ lb. Inserting these values in their proper places in the summary, we readily obtain the values for the assumed maximum shears.

The dead moment at the center due to the girder itself will be $1/8 \times 300 \times 45^2 = 76{,}000$ ft.-lb., and at the point C (see page 64, Fig. 44) will be $3/4 \times 76{,}000 = 57{,}000$ ft.-lb.

The dead moment at D due to the floor-system is found as

follows: The load at D and each point C is equal to the dead shear on the floor beam or 4700 lb. The moment due to this at D is equal to $1.5\times4700\times22.5-4700\times11.25=105750$ ft.-lb. and at C is equal to $1.5\times4700\times11.25=79{,}300$ ft.-lb. The total estimated dead moment at D is then 181,750 ft.-lb. and at C is 136,300 ft.-lb. Inserting these values in their proper places in the summary, we can then obtain the assumed maximum moment at the points C and D. We will now proceed with the design of the girder. As we know what the depth will be, we will use the method given on pages 72 et. seq.

Design of Web. Web Stiffeners. The required thickness of web to resist shear will be $\frac{142010}{10000\times68}=0.21$. We will use 3/8 in. (Spec. ¶ 21, 40). In this connection it should be noted that, as the girder rests on an abutment, the end connection will have no influence in determining the web thickness. There is generally no trouble in getting in a sufficiently close spacing of flange rivets in the girder so that they can properly perform their functions.

The question of web stiffeners should be settled at this point, as it may affect the thickness of the web.

The specifications require the use of the formula (3) page 44 in the case in hand with the following modifications: Stiffeners are required if the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, and these stiffeners must not be further apart than the clear depth of the web nor over 6 ft. apart. It seems unscientific to adopt a formula, and then hedge it about with so many restrictions that the formula itself does not in many cases apply. The New York Central Lines in their specifications give no formula for their stiffeners, but give the following specifications: "The webs of plate girders shall be stiffened with angles at intervals not greater than the depth of the girder nor greater than 5-1/2 ft. Near the ends of the girder, the spacing of intermediate stiffeners shall be about one-half the depth of the girder but shall not exceed 3-1/2 ft. and shall increase toward the center."

"If the unsupported distance between the flange angles is less than 50 times the thickness of the web, intermediate stiffeners may be omitted."

To return to the case in hand, stiffeners are required and mustnot be more than 57 in. apart. The number of stiffeners re-

quired in a panel should be determined. In this case it will be two. These should then be spaced as nearly equidistant as possible. For instance, if a panel is 10 ft. long, and stiffeners are required every 4 ft., do not make the spacing 4, 4 and 2 ft. but make it three spaces of 3 ft. 4 in. each.

The size of the outstanding legs of the intermediate stiffener angles cannot be computed. In deck girders, it is customary to make them as wide as possible without projecting beyond the flange angles (Spec. ¶ 81). The object in this is to minimize the bending of the outstanding leg of the flange angles due to the deflection of the ties under loads.

It should be noted that it is sometimes more economical to increase the thickness of the web to such a point that stiffeners will become unnecessary. This procedure also increases the value of the flange rivets, and may lead to some comparatively small economy there.

It is possible to find out from any of the stiffener formulæ the thickness of web plate which will render stiffeners unnecessary, or the thickness for which a certain spacing of stiffeners will be required. All that is necessary is to insert the given quantities in the chosen formula and solve for the required result. In our case stiffeners are required at intervals of 57 in. As the distance between flange angles is 56 in., the conclusion would naturally be that no stiffeners are needed in this particular case. However, the specifications state (Spec. ¶ 81) that where required because the thickness of web is less than one-sixtieth of the distance between flange angles stiffeners must be used at a distance apart not greater than the clear depth of the web. This will require a spacing of not more than 56 in. in our case.

As the author has stated before, he regards this formula as extremely unscientific because of the many vitiating restrictions placed around it. It would be better to choose a formula in which the restrictions are inherently expressed and apply it to the cases which one is considering. The fact that stiffeners are often required for fabrication can be separately taken care of by stating the spacing required for such purposes. The author has endeavored to obtain a statement from various quarters of the spacing desired for fabrication purposes, but can obtain no information definite enough to warrant quoting.

Forms of Flanges.—The flange angles for this size of girder will probably be 6 in. × 6 in., which will require a 14-in. cover plate.

The cover plates are always made a little wider than the width over all of the flange angles, but not enough wider in the best practice to project more than an inch, or possibly two inches, beyond the edge of the flange angles. In cases where for any reason they do project beyond the flange angles more than 2-1/2 in. a row of rivets should be put through the plates to hold them firmly together.

The specifications do not directly cover this particular case, although ¶ 42 bears upon it somewhat. Where such a row of rivets is used, they should be spaced not further apart than 16 times the thickness of the thinnest plate through which they pass. They should not be computed as carrying any part of the horizontal shear, but are used merely as so-called "stitch-rivets" to hold the parts firmly together. They help to prevent any such distortion of plates as is shown in Fig. 31. If placed too far apart, or omitted entirely, the plates are likely to buckle separately as shown in Fig. 53.

It is considered good design to have at least one-half of the flange area directly connected to the web by rivets. In a flange of the form we are using, this means that one-half the area of the flange will be put into the angles, and the rest will be put into the cover plates. In cases where the largest flange angles are not large enough to make up one-half the area of the flange, it is customary to use side plates as shown in Fig. 59*c*. Various forms of flanges are shown in Fig. 59.

Fig. 59*a* shows the simplest form of plate girder flange.

Fig. 59*b* shows the ordinary form consisting of two angles and cover plates. The cover plates may be of any number, although generally not greater in total area than the flange angles. The cover plates are cut off wherever the moment is small enough to allow it, except that in bridge work the top flange cover plate which is next to the angles usually extends over the full length to prevent water from working in between the flange angles and web. In buildings this is unnecessary as the girders are practically always enclosed.

Fig. 59*c* shows a form used where it would be impossible to obtain enough rivets to connect the angles and web in a flange of the form of 59*b* or where the area of flange required is so great that it is impracticable to put half of it in the angles. As before stated, it is considered desirable to have at least half the flange area directly connected to the web.

Fig. 59 *d*, shows a form of flange used where very heavy sections are required. In cutting off plates the side plates "1" are usually dispensed with first. The angles marked "2" or the cover plates may be dispensed with next. It is generally better

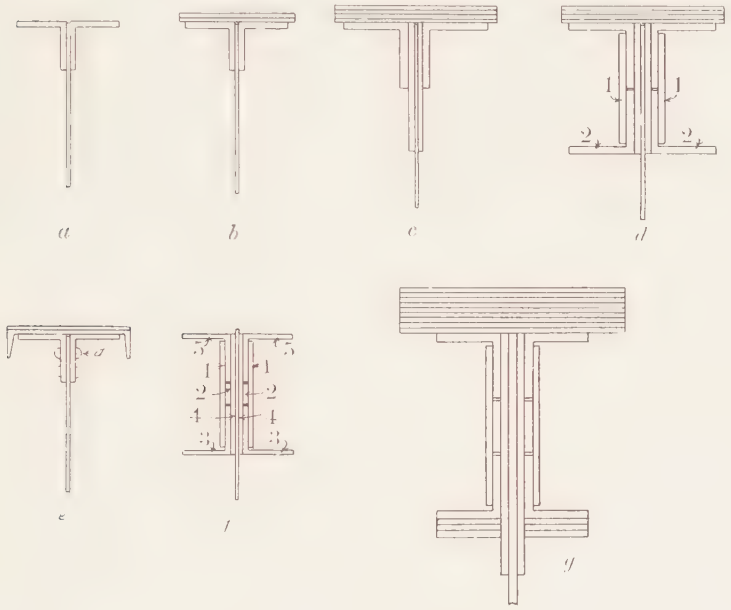


Fig. 59.—*g* is a cross-section of the flange of the heaviest plate girder ever built. It is 120 ft. center to center and 122 ft. 6 in. long over all and weighs 170 tons. The web is 120 in. × 1 in. and the flange is composed at the center of two 30 in. × 1 in. side plates, four 8 in. × 8 in. × 1 in. angles, two 6 in. × 1 in. flats, two 20 in. × $\frac{3}{4}$ in. side plates which are used to splice the various parts of the flange and web where necessary, but are made continuous throughout the length of the bridge for greater simplicity of detail and to avoid crowding of rivets near splices, six 8 in. × $\frac{3}{4}$ in. cover plates on the outstanding legs of the flange angles nearest the neutral axis, and nine 28 in. × $\frac{5}{8}$ in. cover plates. The whole makes up one flange having a gross section, counting the two 20 in. × $\frac{3}{4}$ in. side plates, of 355.5 sq. in. without counting any portion of the web. The reason for using a plate girder instead of a through truss at this point, is that the skew is so great that it would be impracticable to obtain the necessary top lateral bracing. (Fig. 60 is photograph of this girder.)

to cut off the cover plates, for two reasons. One is, that the area of one plate is considerably less than the area of the two angles, and consequently the flange may be kept nearer to the theoretic-

ally required area throughout, and the design will be more economical. The other reason is that the position of the center of gravity of the flange is less disturbed by removing a cover plate than by removing the angles "2." It is desirable not to have too sudden and large changes in the position of the center of gravity of the flange, as it may cause quite large secondary stresses in the girder at the point where the position shifts.

Fig. 59*e* is a form of flange substituting a channel for one of the cover plates. The flanges of this channel should always be turned down so that a pocket for holding water will not be formed. This section is excellently adapted for resisting lateral buckling or sidewise deflection, but is not much used although there does not appear to be any good reason for its lack of use, unless it is the difficulty of getting at the flange rivets *a* in order to rivet up the girder after it is assembled.

Fig. 59*f* shows a form used where a very considerable flange area is required and it is desired to have the web project upward into a notch in the under side of the ties. Plates "2" are shown between the angles in this case. In dispensing with flange area, plates "1" are cut off first, plates "2" next and angles "3" last; plates "4" and angles "5" are continued to the end. In this case the bottom flange may be, and quite commonly is, made of a section similar to 59*c*, if desired, and if sufficient area can be obtained.

Forms 59*d* and 59*f* both give a considerably less effective depth for the same total depth of girder than do the other forms. This type of flange requires a larger section to resist the same moment, but it is the only practicable solution in many cases. Attention is called to the striking similarity between a girder with flanges of the form of 59*d* and a truss with a solid web.

Fig. 59*g* shows a cross section of flange of the heaviest and largest plate girder ever built. It was built in the course of a grade separation project at Worcester, Mass., and carries the tracks of the Boston and Albany over those of the New York, New Haven and Hartford at South Worcester. Fig. 60 is a photograph of the girder before it was erected. One end rests when erected on the shoe shown just above the left hand end. It was designed by W. F. Steffens, then engineer of structures of the Boston and Albany.

Design of Flanges.—The design of the girder flanges is as follows: The top flange may be considered to be supported later-

FIG. 60.

ally at the floor-beams. This support is derived from the gusset plates on the floor-beam, which are extended up to the top flange of the girder as shown on Plate I. The compressive fiber stress then will be

$$16000 - 200\frac{135}{14} = 14072$$

$$\text{The flange stress } \frac{2051750 \times 12}{66} = 373{,}400 \text{ lb. estimated.}$$

	Top flange	Bottom flange
Required area $\frac{373,400}{14,072}$ =	26.54	$\frac{373,400}{16,000}$ = 23.34 sq. in.
Web equiv. 1/8×3/8×68	3.19	3.19
	23.35	20.15
Two 6 in. ×6 in. ×9/16 in. L's	12.88	10.60
	14)10.47	12)9.55
	0.748	0.796
Use one 3/8- and one 7/16-in. plates	0.8125	0.8125

Note that in computing the thickness of plate we use 14 in., the full width of the compression flange and 12 in. the *net* width of the tension flange. This enables us to obtain at once the required thickness of the tension flange as we consider it as a plate of a *net* width of 12 in. The best designers use the same gross area in both top and bottom flanges whenever practicable. In finding the net area throughout, subtract from the gross area an allowance for each rivet hole occurring in the section passing through the greatest number of holes. The allowance made for a rivet hole is for a hole 1/8 in. more in diameter than the diameter of the rivet. The rivet hole is made 1/16 in. larger than the diameter of the rivet, and the other 1/16 in. is allowed for material around the hole which may be injured in punching. In determining the net area some attention must be paid to the proximity of rivet holes in sections adjacent to the one considered. That this is necessary will be understood by referring to Fig. 61, which indicates the way in which a member may fail by tearing diagonally between rivets. In order to prevent diagonal tearing, there must be at least as much area of metal on the most adverse combination of diagonal planes as there is on the net section taken perpendicular to the axis of the member. This object is usually accomplished by making an allowance for these

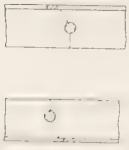


Fig. 61.

adjacent holes based upon their distance from the section under consideration. The most severe specification in this respect with which the author is acquainted is that of the New York Central Lines which is as follows: "The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area, the areas of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane, within a distance of 4 in., and which are on other gage lines than those of the holes cut by the plane, the parts being determined by the formula:

$A\left(1-\frac{p}{4}\right)$, in which

A = the area of the hole, and

p = the distance in inches of the center of the hole from the plane."

The actual effective depth must now be found. This is most easily done by first finding the distance from the center of gravity of the angles to the center of gravity of the whole flange. The computation can readily be arranged in the form of a table as shown below. Dividing the sum of the quantities Ad by the sum of the areas A gives this distance.

Part	Area A	Distance from axis to center of gravity of part d	Ad
2- 6×6×9/16	12.88	0	0
1-14×3/8	5.25	1.71+0.19=1.90	9.98
1-14×7/16	6.13	1.71+0.60=2.31	14.16
.....	24.26	.....	24.14
			0.995

The quantity 0.995 in. may be called 1 in.

Subtracting 1.00 from 1.71 gives 0.71 in. distance of center of gravity of flange from the backs of the angles. The distance back to back of angles is 68.5 in. and the true effective depth then is $68.5-(2\times0.71)=67.08$ in. The flange section must now be revised to see whether or not the true effective depth will affect the design.

Flange stress $\frac{2051750\times12}{67.08}=367{,}500$ lb. actual

Top flange		Bottom flange
Required area $\frac{367{,}500}{14{,}072}=$	26.11	$\frac{367{,}500}{16{,}000}=22.97$

Web equivalent	3.19	3.19
	22.92	19.78
Two 6 in. ×6 in. ×9/16 in. L's	12.88	10.60
	14) 10.04	12) 9.18
	0.72	0.765

Two 3/8-in. plates are not quite sufficient and the design will stand with one 3/8-in. and one 7/16-in. plate in each flange.

It would be well for the student to draw a diagonal line through the first design based on the estimated effective depth in order not to confuse it with the exact final design. The student is cautioned to form the habit of crossing out old figures which do not form a part of the final design, in order to avoid mistakes arising from turning back to the wrong set of figures or computations. As soon as the design of any member is settled upon, it should be noted in the summary in order that it may be referred to readily at any point. This summary should contain in the case of plate girders, (1) the sections of the web and of the top and bottom flanges; (2) the computed flange rivet pitch at as many points as it has been computed; (3) the number of rivets in the hitch angles, end stiffeners, etc.; (4) the required spacing of web stiffeners, if any, and (5) the size of rivets used.

Arrangement of Flange Plates. In the arrangement of flange plates it is quite customary to specify that the thickest plate shall be next to the flange angle, and the thinnest plate on the outside. There seems to be no good reason for this other than the idea held over from constructing crib work that the thickest pieces should be at the bottom and the thickness should gradually decrease, if it changes at all, as the crib work is built up. There is no good reason why, if a flange is made up say of two 1/2-in. and one 3/8-in. plates, the 3/8 plate should not be placed next to the flange angle. In this way it is possible to obtain a more economical design, because it is practically always specified that the plate next to the flange angle shall extend through the length of the bridge in the case of the top flange. The object in this is to prevent water from getting in between the web plate and the flange angles and causing corrosion. It also gives a better finish to the top of the girder and makes it somewhat stiffer in resisting compressive stresses throughout its length. The thinner the plate, then, that is to be next to the flange angles, the greater the economy that will result in the finished design. Also, if it becomes necessary to splice a plate, it can be very

easily done if the next cover plate outside is as thick or thicker than the plate to be spliced. If the outer plate be thinner than the plate to be spliced, it is necessary to add a special splice plate to obtain a sufficient thickness to make the splice of the proper strength. The splices of cover plates should be located, whenever possible, at the point where another cover plate may be dispensed with in order that this plate may be extended and used as a splice plate.

To sum up: The only objection to putting a thin cover plate next to the flange angles seems to be an idea on the part of designers which does not stand careful scrutiny.

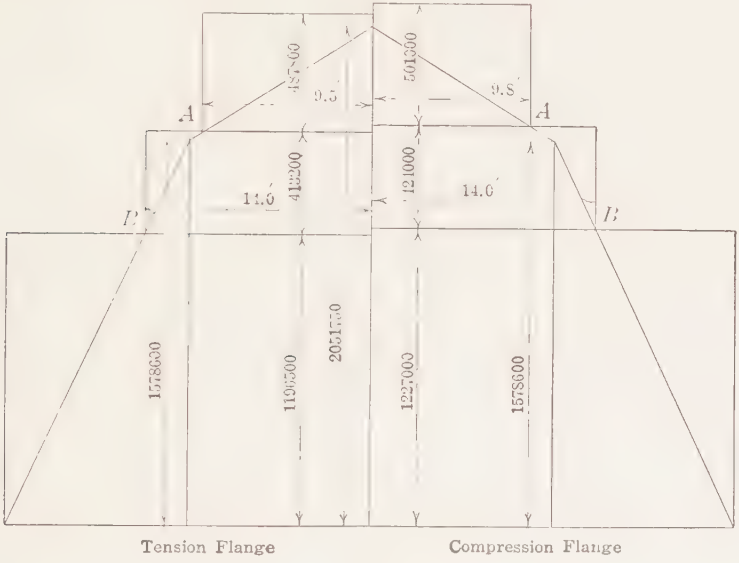


FIG. 62.

Cutting off of Cover Plates.—The next step in the design of the girder is to find the point at which the cover plates may be cut off. This is on the whole best done by plotting a curve of moments on the girder as shown in Fig. 62. It will be noticed that this curve of moments is composed of straight lines between the panel points. It is proper to make these lines straight because the location of loads producing maximum live moments at each of the panel points is usually such that a maximum moment at each of two points will not occur at the same time. The maximum moment at each panel point is plotted on the diagram and these points are connected by straight lines. The

moment obtained from these lines at any point between the panel points will then be somewhat greater than that actually existing at that point. Therefore a diagram made upon this basis will indicate between panel points a required area which is somewhat greater than a strict theoretical analysis would indicate. The left hand side of the diagram will be used for the tension or bottom flange, and the right hand side for the compression or top flange. It is evidently unnecessary in a symmetrical girder to draw a diagram for both ends of either flange. In the cases of unsymmetrical or continuous girders, it is necessary to draw a complete diagram for each girder. A good scale for the student to use in girders of approximately the size of the one under discussion is 1 in. equals 6 ft. for distance, and 1 in. equals 400,000 ft.-lb. for moment. The moment which can be carried by the flange angles and the web equivalent in both the top and bottom flanges should be computed and a line drawn at the proper height in the diagram to represent this moment. The center of gravity of this portion of the flange should be assumed to be at the center of gravity of the angles. Then the moment which can be carried by the first cover plate should be plotted above this and a horizontal line drawn at this height. The moment arm for a cover plate is the distance between the center of gravity of the plate in the top flange and that of the corresponding plate in the bottom flange, leaving out of consideration the flange angles and web. This computation should be made for each successive plate in both the compression and tension flanges. The outermost plate can then be cut off at the point A, and the next plate at the point B, etc.

The computations necessary for finding the moments which the various parts of the flange bear may be tabulated as follows:

A	*B*	*C*	*D*	*E*
Part	Area sq. in.	Allowable stress, lb. per sq. in.	Moment arm, in.	Moment $\frac{B \times C \times D}{12}$ ft.-lb.
Web equivalent and top flange angles.	16.07	14,072	65.08	1,227,000
First top cover	5.25	14,072	68.875	424,000
Second top cover	6.13	14,072	69.69	501,000
Web equivalent and bottom flange angles.	13.79	16,000	65.08	1,196,500
First bottom cover	4.50	16,000	68.875	413,200
Second bottom cover	5.25	16,000	69.69	487,800

It is proper to extend the cover plates beyond the theoretical point at which they may be cut off. The reason for this is that the stress is distributed nearly uniformly over the flange area and consequently wherever a plate forms part of the flange, it carries its proportionate share of the stress. Therefore, at the point at which a cover plate could be dispensed with theoretically, it carries some stress and this stress must be taken out of it and put into the remainder of the flange. The cover plate is extended about a foot for this purpose, and a number of rivets are put through it closely spaced in this extended portion to relieve it of its stress. A little reflection will show that this results in overstressing the rivets somewhat in this extended portion. There is, however, no way of avoiding this difficulty. The cover plates in the top and bottom flanges should be cut off at the same point in order to keep the neutral axis in approximately the same location throughout. Sudden changes in the location of the neutral or gravity axes of sections produce undesirable secondary stresses and should be avoided as far as possible.

Pitch of Flange Rivets.—The required flange rivet pitch in the end and intermediate panels should be computed next. The effective depth of 65.08 in. is that for two angles only, as the cover plates are all cut off in the bottom flange at the end of the girder. There is at this point, of course, a cover plate in the top flange and if desired the exact effective depth at this point computed with one cover plate in the top flange, and no cover plates in the bottom flange, may be used. This, however, is a rather unnecessary refinement as the formula we are using is approximate and at the same time is somewhat on the safe side; certainly far enough on the safe side so that the rivets will not be overstressed when figured in this way.

End panel $\qquad \dfrac{7900 \times 65.08}{142010} = 3.602$

Intermediate panel $\qquad \dfrac{7900 \times 67.08}{68225} = 7.75$

The maximum pitch allowed by the specifications is 6 in. which will be the pitch used in the intermediate panels (Spec. ¶ 41).

End or Reaction Stiffeners.—The design of the end stiffener angles over the abutment comes next in order. These stiffeners are the ones shown at *A* in Figs. 63 and 64. For bridges of the

size we have under consideration two pairs of stiffener angles are generally sufficient as shown in the figure. The entire load which all of these pairs will carry is equal to the total reaction; but the distribution between the different stiffeners is an indeterminate matter. Owing to the fact that the girder deflects, a large proportion of the reaction is carried out by the stiffeners which are at the edge of the abutment nearer to the center of the span.

These are the stiffeners marked *A* in Fig. 63. Some engineers attempt to allow for this deflection by using a tapered or wedge-shaped plate under the end of the bridge. This plate is tapered at such an angle that the sole plate on the girder will bear evenly over its whole length when the bridge has its maximum deflection, or in other words, when it is fully loaded. The maximum reaction is then assumed to be uniformly distributed among the end stiffeners. The accuracy of this assumption is wholly dependent upon the accuracy of the shop work and of the setting of the pedestals and girders. In the case which we have, we will assume the first type of construction with a flat plate under the end of the girder. Under these circumstances, some engineers assume that the whole reaction is taken by the stiffeners *A*, Fig. 63, but this is rather extreme. If we assume that twice as much is taken by these stiffeners as is taken by the end stiffeners, which seems a reasonable assumption, we will have two-thirds of the total reaction or $2/3 \times 142{,}010 = 94{,}670$ lb. on these angles. The function of these angles is to distribute the reaction to the web of the girder throughout its depth by means of the rivets which join them together. The stiffeners then act as columns which are more or less restrained, being entirely restrained in the direction of the length of the girder by the web, and being partially restrained by the stiffness of the web in a direction at right angles to the girder. To allow for the restraint exerted by the web in the latter direction, the length of the angles which is considered as a column is commonly taken as about one-half of the depth of the girder. The bearing area of the stiffener on the bottom flange angle is also one of the elements which must be considered. To determine the width of the outstanding legs we must fulfill the requirements of ¶'s 81 and 19 of the Speci-

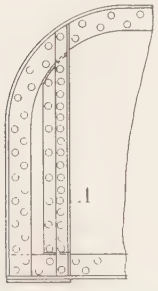


Fig. 63.

fications. As we have a 6-in. flange angle, the outstanding leg of the stiffeners must be as wide as this angle will allow which is 5 in. The 6-in. flange angle has a 1/2-in. fillet at the inner corner, hence the whole of the area of the end of the stiffener cannot be assumed to rest on the bottom flange angle, as the stiffener will be cut or ground off to clear this fillet. The sizes of the fillets in angles is given in the first part of the Cambria hand book with the sections which are rolled by the steel company. See Fig. 64. The length a is $5-0.5=4.5$ in. in this case. The necessary thickness of the stiffener angles in order to give proper bearing area on the bottom flange angles is

$$\frac{94670}{16000\times 2\times 4.5}=0.66 \text{ or } 11/16 \text{ in.}$$

a 5 in. $\times$3-1/2 in. $\times$11/16 in. L is sufficient then so far as bearing on the flange angles is concerned. These angles must also have sufficient strength as columns to carry the reaction (Spec. ¶'s 19, 81). The radius of gyration about which these angles will fail is that about an axis parallel to and in the center line of the web. The stiffeners are completely prevented from failing about an axis perpendicular to the web by the stiffness of the web itself. The least radius of gyration of these angles is

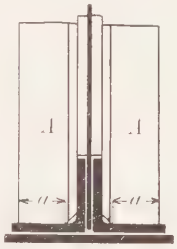


Fig. 64.

$$r^2=1.56^2+(.75+1.70)^2$$
$$r^2=2.45+6.0=8.45$$
$$r=2.9 \text{ in.}$$

Applying the column formula (Spec. ¶ 19) we obtain the maximum allowable compressive fiber stress in the stiffeners as follows:

$$16{,}000-70\,\frac{34}{2.9}=15{,}180$$

Therefore the maximum allowable stress is 13,500 lb. (Spec. ¶ 19). The radius of gyration about the other axis need not be considered, as the web braces the angles thoroughly in this direction.

$$\frac{94670}{13500}=7.00 \text{ sq. in. required in the two angles.}$$

We have 10.76 sq. in. so these angles are sufficient. The whole

area of the angles should be used when computing them as columns, because this area is available everywhere except at the extreme end of the angle. The number of rivets in them must be computed next. Bearing on the web at 7876 lb. gives $\frac{94670}{7876} =$ 13 rivets. As these rivets pass through a filler their number must be increased 50 per cent., if the filler be loose, giving 19 rivets required (Spec. ¶ 60).

By reference to Plate I, it will be seen that with a spacing of 3 in. center to center, it is possible to get 16 rivets beween the flange angles. It will be necessary to use a tight filler at this point. The rivets will be arranged about as shown in Fig. 63. It should be noted that the only reason for using a tight filler is to comply with the specifications. The reason the specifications arbitrarily increase the number of rivets in such a case is to allow for the bending on the rivets which occurs when the loads and reactions on the rivet act through plates which are not in immediate contact. A method for finding the number of rivets to be used in a detail of this kind when a tight filler is shown by computation to be required is given on page 81.

There is no way of computing the thickness or size of intermediate stiffeners in through plate girders, or the rivet spacing to be used in them. It is customary to make these stiffeners as wide as the flange angles will allow, but not to allow the stiffeners to project beyond the edges of the flange angles. The specifications, ¶ 81, provide that the outstanding legs of web stiffeners shall be as wide as the flange angles will allow and shall fit tightly against them. The width of the outstanding legs shall be not less than one-thirtieth of the depth of the girder plus 2 in. This would give as a width a little over 4 in. in our case, thus requiring the use of a 5-in. outstanding leg. One row of rivets is in general all that is necessary in the stiffeners, and these rivets may be spaced at the maximum spacing allowed by the specifications. The thickness of the stiffener angles on through girders is a matter for the judgment of the designer. Except on the very largest girders, they are generally made 3/8 in. thick. As a rule $5\times3\text{-}1/2\times3/8$ intermediate stiffener angles are used with 6×6 flange angles, and $6\times3\text{-}1/2$, or sometimes $7\times3\text{-}1/2\times3/8$ stiffeners are used with 8-in. flange angles. The $7\times3\text{-}1/2$ angle is listed as a special angle, but can often be obtained in the market.

Splicing of Girder Flange.—In girders of ordinary length, it is not necessary to splice the flange angles, and as a rule it is not necessary to splice the flange plates. We will, however, design these splices in order to show how it should be done. In splicing the flange plates, the splices should, if possible, be so located that they will come just beyond the point where the next flange plate outside the one to be spliced may be cut off. By extending this plate it may be used as a splice plate. To make sure that the splice plate will have sufficient cross-section, it is well to arrange the plates so that if there is any difference in their thickness, the heavier plates will be outside and the thinner ones next to the flange angles. The number of rivets required in such a splice is easy to determine. It is not necessary to provide splice rivets in addition to the rivets required to transfer the increment of shear coming from the web to the plates. The explanation given below of splices of flange angles should make this clear. In this connection ¶ 59 of the specifications should be noted.

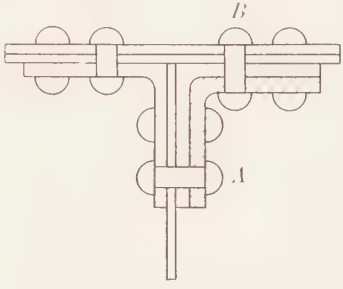


FIG. 65.—Splice angle is cross-hatched.

The splicing of the flange angles should be done by an angle placed as shown in Fig. 65. If only one angle is spliced at a given point, only one splice angle is necessary. To determine the size of the splice angle, it should be so chosen as to have the same net area as the flange angle which it splices. One 6 in.×6 in. ×9/16 in. L has a net area of 5.3 sq. in., taking out two rivet holes. The splice angle will be made from a 6 in.×6 in. angle with the ends of the legs planed or sheared off so that they will be flush with the ends of the legs of the flange angles. The actual length of leg of a 6 in.×6 in.×9/16 in. angle is 6-3/16 in. (See table of overruns on page 273.) The length of the leg of the splice angle will then be 6-3/16−9/16=5-5/8 in. One 7/8 in. rivet is to be taken out of each leg leaving a net length of 5-5/8−1=4-5/8 in. The simplest method of procedure at this point is to assume the thickness of the splice angle and then see whether it has a net area at least equal to the net area of the angle to be spliced. This assumed thickness should be from 1/16 to 1/8 in. more than that of the angle to be spliced. One-eighth of an inch should be

assumed for thick angles and 1/16 in. for the thinner angles. In this case we will assume a 1/16 in. increase in thickness or a thickness of splice angle of 5/8 in. The net area of the splice angle then will be {4-5/8+(4-5/8−5/8)}5/8 or 5.4 sq. in. This area is slightly larger than the angle to be spliced, hence the splice angle assumed is sufficient. The number of rivets each side of the seam should be the number required to develop the angle which is $\frac{16000 \times 5.3}{7200} = 12$. The rivets are evidently limited by their strength in single shear or 7200 lb. It is not generally necessary to put in the full number of splice rivets in addition to those required to carry the shear from the web to the flange angles. The same rivets are available in both cases as they are stressed on different sections. In explanation consider the rivet "A" in Fig. 65. It carries as a flange rivet, stress from the web to both the flange angles. It may be considered to carry a stress to each flange angle from the web equal to the allowable bearing strength of a length of the rivet equal to one-half the thickness of the web, in this case 3/16 in., provided this value is less than the value of the rivet in single shear. This value (if less than single shear on the rivet) may be considered to be unloaded into the flange angle in 3/16 in. of length of rivet. This leaves available 9/16−3/16=3/8 in. of length of rivet to load up with stress to carry to the splice angle. This value is, in this case, greater than the single shear which was used in computing the number of rivets necessary in one side of the splice, hence no extra rivets are needed in the length of splice to carry stress from the web to the flanges.

Another way of looking at the same question is this: At one end of the splice angle let us assume, for the sake of argument, that the flange is fully stressed. At the other end of the splice angle it is evident that, through transfer of stress to the web, the flange is understressed, unless a cover plate happens to end at this point. It follows then that some of the splice rivets, instead of carrying stress to the splice angle from the flange angle, transfer it to the web where it remains. Enough rivets have been provided to carry the whole of the stress from the flange angle to the splice angle. If some of these rivets transfer stress from the flange angle to the web instead of to the splice angle, it makes no difference in the computation of the splice, and it is evidently not necessary to provide rivets to transfer stress to

the web in addition to those provided in the splice under the above conditions. In other words, a rivet in the splice transfers stress from the flange angle either to the splice angle or to the web. It does not matter in which of these directions the stress goes. So long as enough rivets are put in each side of the seam to develop the flange angles, the transfer of stress to the web will be taken care of. The same line of reasoning applies to the rivets "B" in the figure. It is good practice to crowd the rivets closely together in the splice in order to make the splice angle as short as possible. Generally both the flange angles in one flange should not be spliced at the same points, nor should the flange angles on the same side in the top and bottom flanges be spliced at the same point.

Lateral Bracing.—The next step is to design the lateral bracing or wind bracing as it is often called. It is becoming more and more recognized that the real function which lateral bracing fulfills in railroad bridges of ordinary span is that of preventing sidewise deflection under shocks caused by the moving load. This is the reason why in the specifications (¶ 13), the lateral force is specified as a certain percentage of the specified train load on one track, plus an allowance of 200 lb. per foot on each chord for wind. On a plate girder bridge such as we have under consideration, we will then use 400 lb. per foot plus 10 per cent. of the train load. The laterals are then designed as the tension members of a Pratt truss and the necessary number of rivets at the ends determined (Spec. ¶'s 13, 18, 21, 28, 39, 40, 43, 72, 76). The struts or posts of the lateral systems are the bottom flanges of the floor-beams. It is unnecessary to compute the stresses caused in the bottom flanges of the floor-beams by lateral forces as these stresses are compressive and tend to offset the tension due to the bending of the floor-beam under the vertical loads. The bottom flanges of the girders, in the case we are considering, form the chords of the lateral system. One of these chords will be in tension and this tension should be computed, added to that already existing due to vertical loads, and the maximum fiber stress caused by this combination of loads determined (Spec. ¶ 26). If necessary the bottom flanges of the girders should then be increased in area. It should be noted that for stresses produced by longitudinal and lateral forces combined with those from live and dead loads and centrifugal forces, the unit stresses may be increased 25 per cent. above those used for

live, dead, and centrifugal stresses, alone. The reason usually given for allowing these excess unit stresses is that the maximum live and wind loads do not often occur together. There is some doubt in the author's mind whether this reason will bear close scrutiny, because it is becoming generally recognized that the principal lateral forces to which railroad bridges are subjected are those due to side swaying of the train, nosing of the engine, and similar causes. This is recognized in these specifications by stating that the lateral force to be provided for shall be 10 per cent. of the specified train load. These lateral forces probably increase with the speed and consequently with a heavy load at high speed, when the impact allowance should be a maximum, the lateral forces are also a maximum. If the allowance of 10 per cent. of the specified train load is sufficiently large to equal or exceed the possible lateral forces, the allowed increased fiber stress of 25 per cent. will result in stresses which will still be safe. If, however, the actual lateral forces are greater than this allowance, the result may be excessive stresses under certain conditions. These remarks apply only to railroad bridges.

In the case of highway bridges, the lateral force is always assumed to be caused by the wind, and is generally specified as a certain pressure per square foot on the exposed surface of the bridge and upon certain hypothetical loads moving across it. In this case, the use of the wind stress, rather than a percentage of the possible live load, is the only logical method to pursue.

Stresses in Lateral System. Fig. 66 shows the plan of the lateral system. All the diagonals are assumed to be tension members. For the sake of clearness, the members running in one direction are dotted.

The lateral load will be $200+200+(5000\times0.10)=900$ lb. per foot. The lateral load per panel will be $900\times11.25=10{,}125$ lb. The maximum shear in the end panel will be equal to 1-1/2 panel loads or $10{,}125\times1.5=15{,}200$ lb., and the corresponding tensile stress in the diagonal will be $15{,}200\times\frac{17.96}{14}=19{,}500$ lb. The maximum shear in the second panel will be $(1/4+1/2)$ $10{,}125=7600$ lb., and the corresponding tensile stress in the diagonal will be 9750 lb. The tensile stress in the bar c–d is found by passing a section a–a and taking moments about the point b with full loads on the span. This gives $\frac{15200\times11.25}{14}=$

12,200 lb. This stress may occur in either *cd* or *be* depending upon the direction from which the wind is blowing.

Effect of Wind Stress on Girder.—This stress must be added to the maximum flange stress in the girder from live, impact and dead loads. (See page 107.) This sum is 367,500+12,200= 379,700 lb. This total stress must now be divided by the net area of the bottom flange, 23.54 sq. in., which gives a tensile stress of 16,150 lb. per square inch. The meaning of ¶ 26 of the specifications will be clear when it is stated that so long as this stress

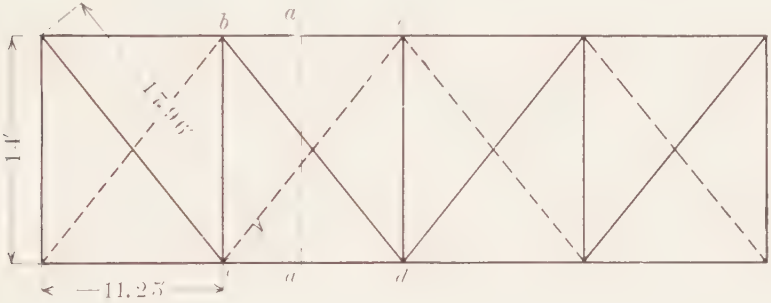


FIG. 66.

(16,150 lb. in this case) does not exceed 16,000+25 per cent. of 16,000, or 20,000 lb. per square inch, the section of the flange will not need to be increased. Should this combination of live, impact, dead, wind, and centrifugal force (if any) cause a fiber stress in excess of 20,000 lb. per square inch sufficient material would need to be added to the flange to bring the fiber stress down to 20,000 lb. per square inch.

Design of Diagonals.—The design of the diagonals is very simple. The required area for the diagonal in the end panel is $\frac{19500}{16000} = 1.22$ sq. in. According to ¶ 76 of the specifications, the smallest angle which can be used is a 2-1/2×3×3/8 in. which gives a net area of 1.95 sq. in. The number of rivets required to connect the laterals to hitch plates will be determined according to ¶ 39 of the specifications which requires that the connections shall have sufficient strength to transmit the greatest stress the member *can* carry. This is a common requirement of good design and should always be followed. A time may come in the life of a structure when an excess of area which happens to exist in a

member will be needed, but it will not be available unless the connections of the member to other parts are strong enough to develop its full strength. The value of a 7/8-in. field rivet in single shear is 6000 lb. and in bearing on 3/8 in. is $3/8 \times 7/8 \times 20{,}000$ lb. or 6550 lb. The lateral can carry $1.95 \times 16{,}000 = 31{,}200$ lb. and will need $\frac{31200}{6000}$ = 6 rivets in its end connection. Where so many rivets are needed in a connection a lug angle would be used. This is a short clip or piece of an angle riveted to the main member near its end and also riveted to the gusset plate as shown in Fig. 67.

A typical lug angle connection is shown in Fig. 67. It is difficult to compute such a connection exactly. In general the field rivets shown in the main angle M should be sufficient to unload the leg which is directly riveted to the gusset. The shop rivets connecting the other leg of the main angle to the lug angle L should be of sufficient strength to transfer the stress in it to the lug angle. The field rivets connecting the lug angle L to the gusset G should be strong enough to transfer all the stress carried by the outstanding leg of the main angle M. In general, also, the center of gravity of the whole group of field rivets (shown in solid black in the figure) should lie as closely as possible upon the gravity axis g_1 of the member M. The gravity axes g_1, g_2, and g_3 of three intersecting members are shown meeting, as they should, at a common point o. The details of the two other members are not shown. A lug angle is not used on Plate I. The lateral connections in the end panel are a little light on this plate.

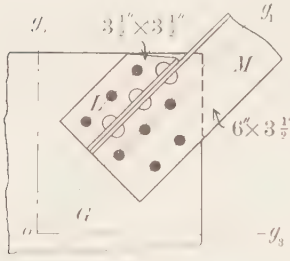


FIG. 67.

Layout of Joints.—Upon the completion of the computations for the lateral system, the layouts of the joints and connections should be made. These layouts of joints and connections should be made on a sheet of duplex paper of large size. The layout should be made like that shown on Plate II. The object of making the layout is so that the spacing of rivets and sizes of plates, etc., around the joints may be scaled instead of calculated. Calculation of these quantities is laborious and unsatisfactory. Lengths of members between intersections should be calculated,

sonry being 600, it is easy to determine how many square inches are required by dividing this into the maximum reaction. This gives $\frac{142010}{600} = 236.7$ sq. in. If it is made 16 in. long, the width will be 15 in. Generally, in spans such as we have under consideration, the sole-plate will be about 14 in. wide and 16 or 18 in. long and 3/4 in. thick. Do not make the length of this plate or casting in odd fractions of an inch. It should be noted that there is a sole-plate, which is riveted to the girder. The rivets in this sole-plate are countersunk on their lower side and chipped off smoothly. This sole-plate rests on the pedestal. The foundation bolts pass up from the masonry through the pedestal and sole-plate and the bottom flange of the girder and have nuts on their upper end. The holes for the anchor bolts in the flange of the girder and sole-plate are round at one end of the bridge and slotted at the other. This slotting is done to allow the bridge to expand. The allowance for expansion is generally 1 in. per 100 ft. length of span (Spec. ¶ 61). The thickness of the pedestal casting or masonry plate can be determined by considering the part of it which projects beyond the sole-plate to be a cantilever beam subjected to a uniform load per square inch equal to the allowable bearing pressure on masonry.

Design of End Stringer.—The design of the end of the end stringer is generally similar to that of the girder. The end of the stringer should be flush with the end of the girder in order that the gap between the end of the stringer and the parapet wall on the abutment may be as small as possible. The end strut, which is intended to keep the stringers in their proper position, is generally made of a built channel section of a depth nearly equal to that of the stringer. The plate connecting it to the girder is made the full height of the girder when practicable. This strut cannot be computed, as it is impossible to find out the proportion of the lateral reaction which goes to each of the girder supports at the same end of the bridge.

SUMMARY OF GIRDER DESIGN

Web 68 in. × 3/8 in.
Flange two 6 in. × 6 in. × 9/16 in. L's—top and bottom.
One 14 in. × 7/16 in. cover 19.6 ft. computed length top and bottom.

One 14 in.×3/8 in. cover, full length, top.
One 14 in.×3/8 in. cover, 28 ft. computed length, bottom.
Two pairs web stiffeners 5 in.×3-1/2 in.×3/8 in. per panel.
Rivet spacing 3.6 in. computed, end panel.
Rivet spacing 7.86 in. computed, center panel.
End stiffeners two 5 in.×3-1/2 in.×11/16 in. L's.
End stiffeners 19 rivets in L's and filler.
End stiffeners 13 rivets in L's alone.

Dead Weight of Girder

Web 68 in.×3/8 in.×19 ft. 2 in.	@	86.7	1661
Web 68 in.×3/8 in.×27 ft. 1-1/4 in.	@	86.7	2350
Two 6 in.×6 in×9/16 in−39 ft. 1 in.	@	21.9	1712
Four 6 in.×6 in.×9/16 in.−8 ft. 3 in.	@	21.9	723
Two 6 in.×6 in.×9/16 in.−46 ft. 4 in.	@	21.9	2030
Two 14 in.×7/16 in.−25 ft. 6 in.	@	20.83	1062
One 14 in.×3/8 in.−37 ft. 2 in.	@	17.85	1656
Two 14 in.×3/8 in.−9 ft. 2 in.	@	17.85	
Two 14 in.×3/8 in.−2 ft. 9 in.	@	17.85	
One 14 in.×3/8 in.−31 ft. 9 in.	@	17.85	
Four 6-1/2 in.×9/16 in.−4 ft. 4 in.	@	12.43	216
Four 5 in.×3-1/2 in.×11/16 in.−5 ft. 4 in.	@	18.3	392
Sixteen 5 in.×3-1/2 in.×3/8 in.−5 ft. 7-3/8 in.	@	10.4	934
Fourteen 3-1/2 in.×9/16 in.−4 ft. 8 in.	@	6.69	437
Two 13 in.×3/8 in.−4 ft. 8 in.	@	16.58	155
Two 3-1/2 in.×3/16 in.−4 ft. 8 in.	@	2.23	21
2600 7/8 in. rivet heads	@	0.2429	632
			13,981

Dead Weight of Laterals

Two 3-1/2 in.×3-1/2 in.×3/8 in.— 7 ft. 9-1/16 in. @ 8.5 lb.
Two 3-1/2 in.×3-1/2 in.×3/8 in.— 7 ft. 9-5/16 in.
Two 3-1/2 in.×3-1/2 in.×3/8 in.—15 ft. 11 in.
Two 3-1/2 in.×3-1/2 in.×3/8 in.— 7 ft. 10-3/4 in.
Two 3-1/2 in.×3-1/2 in.×3/8 in.— 7 ft. 11 in.
Two 3-1/2 in.×3-1/2 in.×3/8 in.—16 ft. 3 in.

2×(63 ft. 6-1/8 in.)	@ 8.5	=1080
Sixteen 5 in.×3-1/2 in.×3/8 in.—9 in.	@ 10.4	= 125
Four 9 in.×3/8 in.—1 ft. 6 in.	@ 11.48	
Two 9 in.×3/8 in.—1 ft. 11-3/4 in.	@ 11.48	=159
Two 9 in.×3/8 in.—1 ft. 11-1/4 in.	@ 11.48	
Four 14 in.×3/8 in.—2 ft. 0-1/2 in.	@ 17.85	=220
Two 14 in.×3/8 in.—2 ft. 1 in.	@ 17.85	
320-7/8 in. rivet heads	@ 0.2429	= 78
		1662

Estimated weight of girder (46 ft. 4 in.)×300=13,900 lbs.

Weight of laterals per panel per girder $\frac{1662}{8}$ =208 lb.

Total dead panel load

Track 200×11.25	=2250 lb.
1 stringer	=1250 lb.
1/2 floor-beam	=1170 lb.
Laterals	= 208 lb.
	4878 lb.

The dead shears on the girder will be as follows:

	From floor	From girder	Total
End panel	3/2×4878=7317 lb.	6991	14,308
Second panel	1/2×4878=2439 lb.	3496	5935

The dead moments on the girder will be as follows:

	From floor	From girder	Total
At center	4878× 22.5=109,750 ft.-lb.	$\frac{13981}{8}$ ×45=78650	188,400
At point C^1	7317×11.25= 82,320 ft.-lb.	$\frac{3}{4}$ ×78,650=58,980	141,300

[1] See Fig. 44.

These values should now be substituted in the table of stresses on page 64 and the actual values of the shear and moments computed and the design revised, if necessary. The difference between the assumed and actual shears is small; so small that revision is evidently unnecessary. The design of the flanges was quite close and it should be investigated to see whether the additional dead moment affects it. This can readily be done by finding the additional area required in the tension flange to carry the difference between the actual and the assumed moment. The tension flange is considered because its design was closer than the compression flange. See page 108. Additional area required $\frac{5000\times12}{67.08\times16000}=0.056$ sq. in. We have $12\times0.0475=0.57$ sq. in. to spare in the tension flange as designed and so the section is all right.

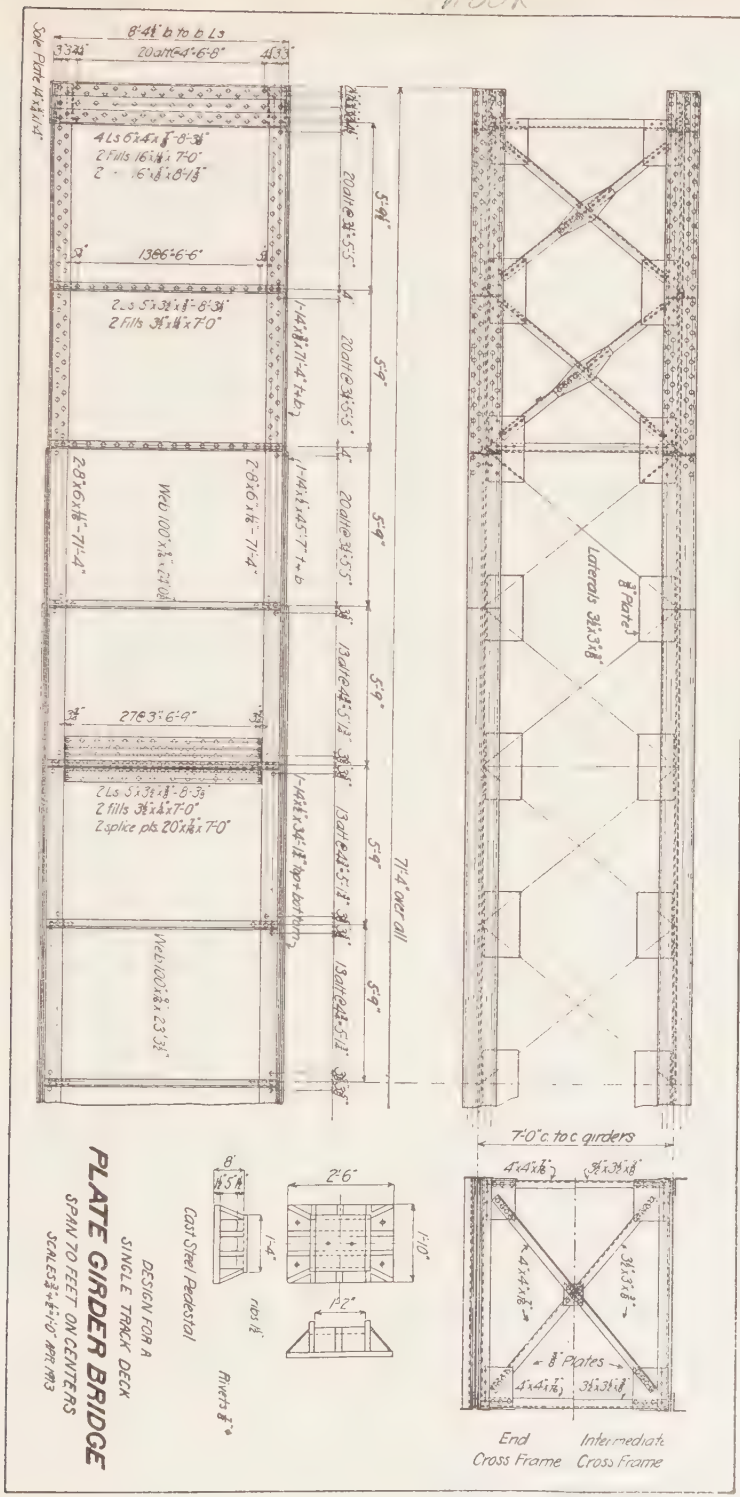
CHAPTER V

DECK PLATE GIRDER DESIGN

Loads.—In designing a deck plate girder bridge, the steps are similar to those used in the through plate girder already worked out. The live moment is known, and the dead moment must be assumed in accordance with the best information available, or must be based on the designer's judgment. For plate girder bridges the dead load is in general such a small proportion of the total load that errors in it are not of the same relative importance as errors in larger bridges, and require less extensive correction to be made in the computations. The weight will depend to a considerable extent on the depth chosen for the girder, and also upon the specifications followed in making the design. The depth will in turn depend largely upon local conditions, which are very variable. For all these reasons, it seems scarcely worth while to attempt to make curves or tables which will fit all cases.

Determination of Depth. Economic Depth.—The first step in the design is to determine upon the depth to be used. Often this is limited by clearances, and in such cases, the depth being fixed, the rest of the steps in the design are straightforward and simple. When entirely free to choose the depth, as might be the case on a high fill, a viaduct, or over a narrow and deep gorge, it is possible to choose the economic depth. The economic depth may be defined as the depth of girder for which the minimum amount of material will be required. Many attempts have been made to deduce a formula for the economic depth with, it must be confessed, no great amount of success. With a girder composed of merely a web and flange angles with no web stiffeners, the finding of the economic depth becomes a simple matter. Under these circumstances it will be that depth at which the minimum allowable size and thickness of flange angle is just sufficient to carry the maximum moment in combination with the web. Where the moment to be carried is large, this will result in depths which are beyond practical limits both of fabrication and shipment.

Where the moment is great enough to require more than a



(Facing page 126.)

moderate thickness of angle in the flange with a reasonable depth of girder, it is very desirable to use cover plates for part of the flange section, because the section can then be varied at different points along the girder to more nearly meet the exact requirements as to flange area. This will result in an economy of material, as a considerable amount of material is wasted toward the ends of the girder if the flange is kept of the same section throughout its length. By varying the section to fit the moment, a large proportion of this material can be saved. The spacing of the web stiffeners must also be considered, as an increase in the depth increases the length of the web stiffeners without correspondingly decreasing their number. The spacing of web stiffeners is by no means susceptible of exact mathematical treatment. There are various formulæ which purport to give the required spacing. They are not accurate, as they depend altogether upon the assumptions made in deducing them. For instance, in the case of the through plate girder which is designed in Chapter IV, the stiffener spacing varies from 22 in. to 57 in., depending solely on the formula used. The stiffener formula which is chosen will evidently have a great influence upon the economic depth. Some studies which the author has made indicate that the economic depth of the usual form of plate girder is almost entirely a function of the stiffener formula used. Consequently, an attempt to deduce an exact formula for the economic depth is wellnigh a hopeless one. Certain studies of railroad plate girders using the *E*-60 loading seem to indicate that about one-eighth of the span is the most economical depth when designing under the American Railway Engineering Association's specifications. The depth of the girder should not be less than one-twelfth of the span except under very exceptional conditions. Depths less than this require very heavy flanges in order to avoid undue deflection and are, as a rule, quite uneconomical.

Considerations of stiffness, and consequently deflection, have an important bearing on the depth of girders. Wherever possible girders should be of sufficient depth to keep the deflection within proper bounds and at the same time utilize the highest practicable fiber-stress in the material of which they are composed.

DESIGN OF 70-FT. DECK RAILROAD GIRDER

Depth.—In the 70-ft. girder we are designing in this chapter, one-eighth of the span would give a depth of 105 in. We will

take a depth of 100 in. for the web. The deflection with these assumed dimensions will not exceed the value obtained by using formula II, page 58

$$100 = \frac{5}{24} \times \frac{70 \times 12 \times 16000}{30000000 \times Z}$$

$$Z = \frac{93}{100000} \text{ or the deflection will not exceed}$$

$$\frac{93}{100000} \times 840 = 0.78 \text{ in.}$$

The live moments and shears are for the loading known as Cooper's *E*-60 (see Table I), and the dead load has been assumed as 575 lb. per foot for each girder, and 400 lb. per foot for the track. These moments and shears are given below at 5-ft. intervals from the center to the end in units of thousands of foot-pounds.

	Center							End
Live	2561	2500	2355	2093	1756	1295	722	0
Impact	2080	2030	1910	1695	1424	1050	585	0
Dead	475	465	436	388	326	232	126	0
Total	5116	4995	4701	4176	3506	2577	1433	0
Parabolic	5116	5010	4700	4170	3440	2500	1360	0

The line of figures marked "parabolic" gives the moments at 5-ft. intervals from the center to the end, assuming the same center moment, and a variation in accordance with a parabolic curve from there to the end.

The maximum shears are as follows in pounds:

	End	Quarter-point
Live	165,800	98,900
Impact	134,400	82,500
Dead	27,200	13,600
Total	327,400	195,000

The steps in the design are similar to those followed in designing the through girder in Chapter IV.

Design of Web.—The web thickness should be computed first and is

$\frac{327400}{10000 \times 100} = 0.327$ in. required to carry shear. The minimum

allowable thickness of material is 3/8 in., and the thickness is also further limited by the specifications. By ¶ 30 of the specifications, the minimum thickness of plate girder webs is 1/160 of the clear distance between flange angles. This gives a thickness of 9/16 in. for the web. The reason for this requirement is to prevent the use of extremely deep and thin webs. The method of determining the thickness of web by shear alone, leads to very thin webs in the great majority of cases. Very deep and thin webs require a considerable number of web stiffeners. Thin webs also require closer spacing of flange rivets. The required spacing of web stiffeners for this thickness should now be computed, as it may be found desirable to increase the thickness of the web in order to reduce the number of web stiffeners. The formula to be used is that of paragraph 81 of the Specifications altered to the form given on page 44, which is, $s = 12{,}000t - 40d$. The effective depth may be assumed as 98 in. which is 2 in. less than the depth of the web.

$$\frac{327400}{98} = 12{,}000 \times 9/16 - 40d$$

$$3340 = 6750 - 40d$$

$$-3410 = -40d$$

$$d = 85 \text{ in.}$$

Stiffeners are required at a spacing not to exceed 6 ft. by the specifications, so the web thickness chosen is satisfactory.

Author's Rule for Web Thickness.—For ordinary proportions, that is, for girders whose depth is from one-eighth to one-twelfth of the span, the following rule for assuming the web thickness will be a fairly good guide in the absence of any specific limiting provision such as occurs in paragraph 30 of the New York, New Haven & Hartford Railroad Specifications. Make the thickness of the web in sixteenths of an inch equal to the depth of the web in feet, with a minimum thickness of 3/8 in. in railroad and 5/16 in. in highway bridges and architectural work. It will be seen that this rule gives the same result as the one in the specifications in this case. It does not give sufficient thickness in cases of through double-track girders where one girder carries a whole track and its load. In such cases the thickness will probably be determined by the bearing value required of the flange rivets when using the minimum pitch.

Effect of Flange Rivets on Web Thickness.—The necessary thickness of web for the minimum allowable pitch of flange rivets should generally be computed, as this may materially influence the design of the flange under some conditions. For instance, if a very thick web were required to give sufficient value to the flange rivets with the usual type of flange (Fig. 59*b*), it might be necessary to use side plates in the flanges as shown in Fig. 59*c*, in order to obtain a sufficient number of rivets to connect the flange and web, using a reasonable thickness of web. The minimum rivet spacing for a 6 in. $\times$6 in. angle is 2 in. in two lines, allowing the rivets to be 3 in. center to center and using a gage of 2-1/4 in.

The usual specification for minimum spacing of rivets is that they shall be not less than three diameters of the rivet apart. For 7/8-in. rivets this will give a spacing of 2-5/8 in. In many cases it is stated that the minimum spacing for 7/8-in. rivets shall preferably be 3 in. The spacing equal to three diameters is often too close and should be avoided. Rivets spaced so closely in two rows are especially bad on account of the liability to diagonal tearing between the holes (see Fig. 61). This is generally taken care of by a clause in the Specifications which requires holes in sections through successive rivets to be considered as occurring, in part at least, in the same section if they are closer together than a certain stated minimum (see page 106). By the approximate method, then, one rivet must carry at least $\frac{327400\times 2}{98} = 6682$ lb. The required thickness of web to give this rivet value is $\frac{6682}{7/8\times 24000} =$ 0.32 or 5/16 in. This is less than the minimum and consequently the 9/16-in. web is sufficient.

Design of Flanges.—We will now proceed with the design of the flanges of the girder. The effective depth is assumed to be 98 in., which is 2 in. less than the depth of the web. The maximum flange stress equals $\frac{5116000\times 12}{98} = 626{,}000$ lb. In order to determine the allowable stress in the top, or compression, flange, we must know its unsupported length. It may fairly be assumed that the lateral bracing will run at an angle of approximately 45 degrees with the axis of the bridge. This will give an unsupported length of twice the distance between girders, or 7 ft. If 8$\times$8 in. flange angles are used, the cover plates will be 18 inches wide for a deck girder. It is undesirable to use very wide flanges when the ties

rest directly upon them as in this case, unless the center of the rail is directly over the web of the girder. The reason for this is that the ties deflect a certain amount under the load on the rail and consequently have a tendency to bear more heavily on the edge of the girder toward the rail. This has the effect of bending the flange sidewise as shown on an exaggerated scale in Fig. 51, page 74. This condition is worse with wide flanges than with narrow ones, consequently it is considered better design to keep the top flange as narrow as is consistent with obtaining sufficient bearing area between the ties and the girder, having due regard to the reduction of allowable compressive fiber stress which necessarily accompanies narrow flanges. We will therefore use 8-in. × 6-in. angles with the 8-in. flange against the web.

	Top flange	Bottom flange
@ 16,000 − 200 $\frac{7\times12}{14}$ = 14,800		@ 16,000 lb.
Required area	$\frac{626{,}000}{14{,}800}$ = 42.30 sq. in. gross	$\frac{626{,}000}{16{,}000}$ = 39.13 sq. in. net
Web equivalent $\frac{100}{8}\times\frac{9}{16}$ =	7.03	7.03
	35.27	32.10
Two 8×6×11/16-in. L's	18.30	15.55
	14)16.97	12)16.55
	1.21	1.38
One 14×3/8 plate	19.25 sq. in.	16.50 sq. in.
Two 14×1/2 plates		

The actual effective depth must now be found. This is most readily done by taking the moment of the cover plates about the center of gravity of the flange angles and dividing by the combined area of the angles and plates. Area of plates 1.375×14 = 19.25 sq. in.

$$\frac{19.25\times(2.54+0.69)}{19.25+18.30} = 1.65 \text{ in.}$$

2.54 − 1.65 = 0.89 or the center of gravity of the flange is 0.89 in. inside of the back of the angles. It is possible to obtain a sufficient section of fairly good proportions in this case by using 6×6 angles instead of 8×6. The thickness would have to be 15/16 in. in order to have about half the flange area directly con-

nected to the web. This thickness is so great that it would be desirable to drill the rivet holes out of the solid material, instead of punching them, even if it were not required by the specifications. It would also require considerably more material in the fillers under the web stiffeners and would lead to a waste of material there. One great objection to using 6×6 angles in this case is that so much of the material would of necessity be put into the cover plates that the center of gravity of the flanges would be outside of the web plate. This would result in secondary stresses, which are more fully discussed in the chapter on plate girder theory. The effective depth then will be $100.5-1.78=98.72$ in. The distance back to back of angles should be made 1/2 in. greater than the depth of the web in order that the irregularities always present in the web may not project above the backs of the flange angles, and so require chipping off before the cover plates can be put on.

The flange must now be redesigned using the new effective depth.

Flange stress $\frac{5116000\times12}{98.72}$ 621,900 lb. This reduction in flange stress will reduce the required area to $\frac{621900}{16000}=38.87$ in the tension flange. This will reduce the required thickness of plates in the tension flange by an amount equal to

$$\frac{39.13-38.87}{12}=0.021 \text{ in.},$$

which is enough to make up for the slight shortage of area as computed in the first design on page 131.

Spacing of Flange Rivets.—The required spacing of flange rivets should be computed next. At the end, the horizontal shear to be transferred from the web to the flanges in 1 in. is $\frac{327000}{97.64}=3380$ lb. There is also a vertical component from the weight resting on the top flange. This may be taken as one wheel load, with 100 per cent. impact, distributed over three ties or about three feet and will equal $\frac{30000\times2}{36}=1600$ lb. per inch of length. It is unnecessary to consider the dead weight of the track, as it is so small in comparison with the live load. The resultant will be

$$3380^2 = 11{,}424{,}400$$
$$1600^2 = 2{,}560{,}000$$
$$3740^2 = 13{,}984{,}400$$

As the value of one rivet is limited by bearing on the web or 11,800 lb. the required pitch at the end is $\frac{11{,}800}{3740} = 3.16$ in., say 3-1/4 in. As the flange angle is 8×6, it will be necessary to use two rows of rivets. It should be noted that three rows may be used in an 8-in. angle leg when necessary. The required rivet spacing at the quarter point is found by a similar process as follows:

$$\frac{195000}{98.72} = 1980 \text{ lb.}$$

$$1980^2 = 3{,}920{,}400$$
$$1600^2 = 2{,}560{,}000$$
$$2540^2 = 6{,}480{,}400$$

$$\frac{11080}{2540} = 4.36 \text{ or } 4\text{-}3/8 \text{ in.}$$

The rivet spacing may be computed at other points if it is desired to make it conform closely to the theoretical.

Spacing of Web Stiffeners.—The spacing of web stiffeners is the next step. This spacing has already been computed to be 85 in. from the formula of paragraph 81 of the Specifications, which also fixes the maximum limit at 6 ft. (See page 100 in this connection.) We will then space the stiffeners 6 ft. apart. In a deck girder, the stiffeners help to prevent the deflection of the flange under the ties and are desirable for this purpose if for no other. The spacing required for this is very uncertain, and is practically impossible to arrive at theoretically. The thickness of the stiffener may be determined on the assumption that the greatest load that will be transmitted to the web by one pair of stiffeners is one wheel load, or 30,000 lb. in this case. With a 100 per cent. allowance for impact this will give a total force of 60,000 lb. to be cared for. The radius of the fillet in the 8-in. $\times$ 8-in. flange angle is 5/8 in. (in this connection see page 113). The net length of one 6-in. angle leg which bears on the flange will therefore be 5-3/8 in. The required thickness will then be $\frac{60000}{16000 \times 2 \times 5.375} = 0.349$ in., or a 3/8-in. angle will be sufficient.

The function of the stiffener is to transmit a wheel load to the web of the girder through the medium of the rivets connecting the two. The required number of rivets to connect the stiffener to the web will be 60,000 divided by the value of one rivet in bearing on the web or double shear, whichever is the less. In this case bearing on the 9/16-in. web limits at 11,800 lb. The number of rivets required will be $\frac{60000}{11800}$ or 6. This must be increased by 50 per cent. according to the Specifications (¶ 60) because these rivets pass through a loose filler. The object of this is to allow for the bending supposed to exist on a rivet when it extends through a loose plate between the parts from which it receives its load. In any case the rivets will not be spaced more than 6 in. apart.

End or Reaction Stiffeners. The design of the end stiffeners is similar to that for the through plate girder (see page 111), except that the full depth between flange angles is available for rivets as the top corner is not curved. Assuming a 5-in. $\times$3-1/2-in. angle, the computation is as follows:

$$\frac{2}{3}\,\frac{327400}{16000\times 2\times 4.375} = 1.57 \text{ in. or } 1\text{-}9/16 \text{ in.}$$

required thickness of stiffener angles.

Fig. 68.

The required thickness of stiffener angles is greater than can be obtained. There are several ways out of this difficulty. One is to use three pairs of stiffeners at the end instead of two. Another is to use a filler under the stiffener and over the flange angle equal in thickness to the fillet on the angle (see Fig. 68). This makes it possible to count upon the whole area of the end of the stiffener bearing upon the flange angles. In such a case the stiffener angles would be limited by their strength as a column rather than by the bearing value of their ends. The maximum allowable stress which they can carry acting as a column is 13,500 lb. per square inch. The required area of the two angles is $\frac{2}{3}\times\frac{327400}{13500} = 16.15$ sq. in. As the whole area of the angle is utilized in bearing, a pair of 6-in.$\times$4-in.$\times$7/8-in. angles can be used with the 6-in. leg against the web. These angles are 0.18 sq. in. too small, but this is

permissible in this case. This type of detail requires long rivets through the end stiffeners. In general long rivets should be avoided as much as possible. The reason for this is that the rivets are smaller than the hole in whch they are driven. They are of course driven hot and must be upset (*i.e.*, increased in diameter) until they fill the hole. This is done by the action of the riveting machine. The longer the rivet is the greater the likelihood that it will not properly fill the hole. In some cases where the use of long rivets is unavoidable, they are made of a tapered form, being larger near the head than they are at the end. Less upsetting action is required on such a rivet, and it is claimed that they fill the hole better. Most specifications take cognizance of the difficulties in properly driving long rivets by increasing the number of rivets when their "grip" (the aggregate thickness of material through which the rivet passes) exceeds a certain multiple of the rivet diameter. (See Spec. ¶ 44.) The grip of the rivets in our case will equal $\frac{9}{16}+2\times\frac{11}{16}+2\times\frac{5}{8}+2\times\frac{7}{8}=4\text{-}15/16$ in. This exceeds 4 diameters (3-1/2 in.) by 1-7/16 in. and the computed number of rivets must be increased 23 per cent.

The computed number of rivets will equal $\frac{2}{3}\times\frac{327400}{11800}=19.$ Since these rivets pass through a loose filler we must add 50 per cent. (Spec. ¶ 60) making 28 in all. This number must now be increased further by 23 per cent., making a total number of rivets equal to 35 in the stiffeners. If the filler is made tight by passing the additional 50 per cent. of rivets required by the specifications through the fillers only, the grip of these rivets will be reduced to less than four diameters and only the 19 rivets passing through the angles will need to be increased 23 per cent. in number. To put the additional rivets through the filler only, it is necessary to widen the filler at least 3 in. more than the width of the angle leg in contact with it, to allow of a row of rivets being driven in this additional width. The fillers in this case would be made wide enough to pass under both pairs of end stiffeners. It is hardly necessary to say that the additional rivets are never put under the stiffeners and countersunk.

Pedestal.—The pedestals used at present are nearly always cast steel. The area in contact with the girder is required to be $\frac{327400}{16000}=204.6$ sq. in. (Spec. ¶ 19). The area in contact with

the masonry is required to be $\frac{327400}{500} = 655$ sq. in. If the bearing of the pedestal on the bottom of the girder be made 16 to 18 in. long, ample area will be provided for the transfer of stress from the girder to the pedestal. The length of the pedestal measured parallel to the girder will depend somewhat upon the available width of bridge-seat. If we assume that it can be made 22 in. long, we will have a width necessary of $\frac{655}{22} = 29.8$ in. or 30 in. The dimensions of this pedestal will be as shown on Plate III. The necessary height may be determined by considering the portion of the pedestal outside of the girder as a cantilever beam with a load pressing upward on the bottom equal to 500 lb. per square inch. Generally the ribs are sloped downward at an angle of about 45 degrees with the horizontal. The portion between the ribs may be considered as a slab supported on the ribs.

Cutting off of Cover Plates.—(See Fig. 69.) The points at which to cut off cover plates may be determined by a process similar to that used on the through plate girder of Chapter IV. The diagram of moments is drawn by plotting the maximum live and dead moments at 5-ft. intervals and drawing a smooth curve through the points so determined. This curve will be almost exactly a parabola (see table on page 128), and therefore a ready analytical method for cutting off plates when the moment curve is a parabola may be applied. As the moment curve is parabolic, with its vertex at the center of the span, the reduction in moment, and consequently the reduction in required flange area, will vary directly as the square of the distance from the center of the span divided by the square of the half-span.

Let A = the area of the whole flange at the center.

A_1 = the total area to be cut off between the center of the span and the point d_1.

d = the half span.

d_1 = distance to point where cover plate may be dispensed with.

Then $$\frac{A_1}{A} = \frac{d_1^2}{d^2} \text{ or } d_1^2 = \frac{A_1 d^2}{A}$$

Since A and d are constants in any given case, a single setting of the slide-rule will enable one to determine where to cut off all the plates on a span.

In our case the process would be as follows for the bottom flange.

For outer cover plate $d_1^2 = \frac{A_1 35^2}{39.08}$ or $d_1^2 = \frac{6 \times 35 \times 35}{39.08}$ $d_1 = 13.7$ ft.

For second cover plate $d_1^2 = \frac{12 \times 35 \times 35}{39.08}$ $d_1 = 19.4$ ft.

For third cover plate $d_1^2 = \frac{16.5 \times 35 \times 35}{39.08}$ $d_1 = 22.7$ ft.

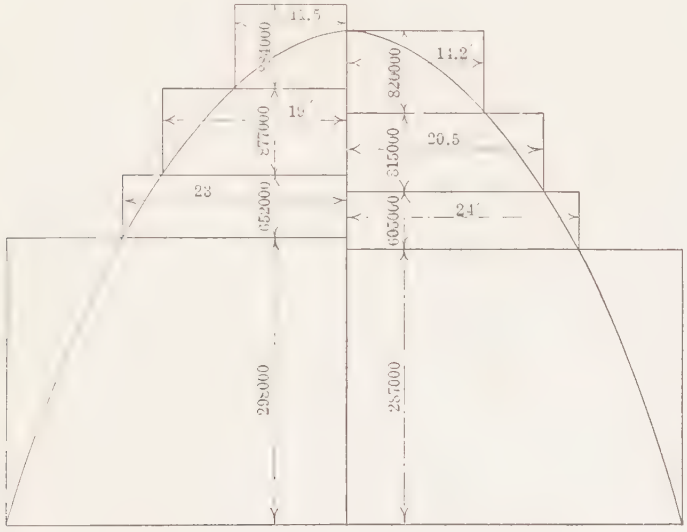


FIG. 69.

As will be seen these distances agree fairly well with those determined by using the curve. This method is close enough in most cases involving railroad deck girders when we consider that we add a foot to each end of the plate to unload the stress in it as explained in Chapter IV, page 111.

Lateral and Sway Bracing.—The length of panel to be used in lateral and sway bracing must be assumed. The sway bracing must be connected to the web stiffeners for economy. As these occur 6 ft. apart we will divide the span up into 6-ft. lengths as nearly as practicable and space the web stiffeners uniformly from end to end. Allowing 18 in. for the length of the end bearing we will have a clear distance between end stiffeners of $70 - 1.5 = 68.5$ ft. This will need to be broken up into 12 parts

in order that no space shall exceed 6 ft. This gives a panel length of $\frac{68.5}{12} = 68\text{-}1/2$ in. This length should not be figured any finer than to the nearest 1/2 in., as it is not desirable to space the rivets in odd 16ths of an inch when possible to avoid it. The girders are to be spaced 7 ft. apart on centers, as this gives good proportions. In general a width of one-tenth of the span center to center for plate girders of ordinary spans gives satisfactory results; and the girders should not be less than 6 ft. 6 in., nor more than 8 ft. or 8 ft. 6 in. center to center. Sway or cross frames should be used at every other stiffener. A single system of bracing in the plane of the top and another one in the plane of the bottom flanges is often sufficient. The load to be provided for is equal to $0.1 \times 6000 = 600$ lb. per foot, to which must be added 200 lb. per foot, making 800 lb. per foot for the loaded chord. The unloaded chord must carry 200 lb. per foot, and the loads must be considered as moving. (Spec. ¶ 13.)

The length of each diagonal equals 9 ft. 3/8 in. The ratio of stress in diagonal to stress perpendicular to girder is

$$\frac{9 \text{ ft. } 3/8 \text{ in.}}{7 \text{ ft.}} = 1.29$$

The maximum load on each 68-1/2-in. length equals $\frac{68.5}{12} \times 800 = 4570$ lb. for the loaded chord. This load may be applied either on *e f* or *g h* (Fig. 70). Using the approximate method, the maximum stress in the different bars is as follows:

Diagonal bar		
6–7	$\frac{21}{12}$	$\times 4570 \times 1.29 = 10{,}300$
7–8	$\frac{28}{12}$	$\times 4570 \times 1.29 = 13{,}800$
8–9	$\frac{36}{12}$	$\times 4570 \times 1.29 = 17{,}700$
9–10	$\frac{45}{12}$	$\times 4570 \times 1.29 = 22{,}100$
10–11	$\frac{55}{12}$	$\times 4570 \times 1.29 = 27{,}100$
11–12	$\frac{66}{12}$	$\times 4570 \times 1.29 = 32{,}400$

These expressions are made up as follows

$$\frac{1+2+3+4+5+6}{12} = \frac{21}{12} \text{ and } \frac{1+2+3+4+5+6+7}{12} = \frac{28}{12} \text{ etc.}$$

A little thought will show that they give the maximum shear on a section passing through the bar which is being comp ted. By setting the constant quantity $\frac{4570 \times 1.29}{12}$ on the slide rule all the stresses may be read off at once. These stresses may be either tension or compression depending upon which way the wind blows. The member must be designed to carry both stresses (Spec. ¶ 23). In laterals the minimum size angle allowed is 3-1/2 in. × 3 in. × 3/8 in. (Spec. ¶ 76). In designing, therefore, we should begin with the lateral having the greatest stress and design the members successively until we arrive at one requiring the minimum size. Beyond this point the minimum size can be used without computation. The ratio of length to least radius of gyration must be considered in this connection, as all of the members are subjected at times to compressive stresses. This ratio

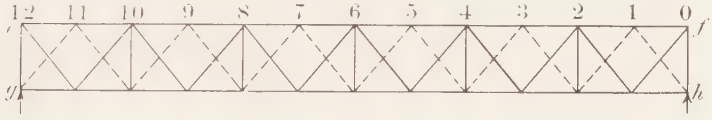


FIG. 70.

must not be more than 120 (Spec. ¶ 22*a*). The length of the laterals is 108 in. and this divided by 120 gives 0.9 in. as the least radius of gyration permissible. The least radius of gyration of a 3-1/2 in. × 3 in. × 3/8-in. angle is 0.62 so that it is too small. A 6-in. × 6-in. × 3/8-in. is the smallest angle having a radius of gyration in excess of 0.9.

This angle is so large that it will evidently be better to use a double system of bracing, as shown in Plate III. The laterals are connected where they cross and consequently the unsupported length may be considered to be reduced to 54 in. The stress may be assumed to be divided equally between the laterals in any one panel. The least radius of gyration which we can have is then $\frac{54}{120} = 0.45$ or the minimum size of angle (3-1/2 × 3 in. × 3/8 in.) can be used. It has a least radius of gyration of 0.62, and $\frac{l}{r} = \frac{54}{0.62} = 87$. The allowable compressive unit stress for this ratio is $16{,}000 - 70 \times 87 = 9910$ lb. and the total stress the angle can carry is $9910 \times 2.30 = 22{,}800$ lb. As the maximum compression on

diagonal 11–12 is now $\frac{32400}{2} = 16{,}200$ lb. the minimum size of angle can be used throughout.

The end sway-frame (see Plate III) must be designed to carry all the lateral loads from the top lateral system to the abutment. The horizontal component of the maximum load on this frame equals $800 \times \frac{70}{2} = 28{,}000$ lb. The length of the diagonal equals 10 ft. 10-5/8 in. $(7 \text{ ft.})^2 + (8 \text{ ft. } 4 \text{ in.})^2 = (10 \text{ ft. } 10\text{-}5/8 \text{ in.})^2$. The required net area of the diagonal will be $\frac{(10 \text{ ft. } 10\text{-}5/8 \text{ in.})}{(7 \text{ ft.})} \times \frac{28000}{16000} =$ 2.72 sq. in. A 4-in.×4-in.×7/16-in. angle is sufficient. All the angles in the end sway-frame will be made 4 in.×4 in.×7/16 in. The intermediate sway frames will be made of 3-1/2-in.×3-in.× 3/8-in. angles. There is no way of computing these intermediate frames unless some assumption as to the stress they may carry is made. The number of rivets in the lateral angles must be computed next. The last sentence of ¶ 23 of the Specifications makes it necessary to compute the connections for the sum of the tensile and compressive stresses which they may carry. This computation is best arranged in a table as shown below. (See Spec. ¶ 76.)

Bar	Doubled stress	Value of one field rivet single shear	No. of rivets
11–12	32,400	6000	6
10–11	27,100	.....	5
9–10	22,100	.....	4
8–9	17,700	.....	4
7–8	13,800	.....	4
6–7	10,300	.....	4

The six rivets needed in the end panel will require the use of lug angles (see Fig. 67). The bottom lateral system, however, is like the top because the top system is constructed using minimum size angles. It is, however, subjected to only about one-quarter of the stress which the top system carries. It is, therefore, good judgment, as well as good design, to assume that the intermediate cross frames carry enough load to the bottom lateral system to make four rivets a sufficient connection in any part of the top lateral system. The great advantage in making this assumption lies in the possibility of making all panels of both lateral systems alike. The maximum stress in the girder flange due to the lateral forces is equal to $\frac{800 \times 70 \times 70}{8 \times 7} = 70{,}000$ lb.

This stress must be added to the flange stress already existing and should be treated as a compressive stress because the top or compression flange also gets the maximum lateral stress. The total flange stress then is $621{,}900+70{,}000=691{,}900$ lb. and the fiber stress will equal $\frac{691900}{44.58}=15{,}520$ lb. per square inch. According to the Specifications (¶ 26) we may use a fiber stress under these conditions equal to $1.25\times14{,}800=18{,}500$ lb. per square inch and therefore need not increase the section of the flange.

Splice of Girder Web.—The girder web is too long to be obtained in one piece, and in fact two splices will probably be needed. If this splice is made strong enough to take care of the maximum bending moment the web can carry, together with the maximum shear to which the girder is to be subjected, it can be located anywhere in the length of the girder. We will proceed as was done in the splice of the floor-beam web in the design of the through plate girder, except that it is unnecessary to compute the horizontal distances to the rivets because the splice is so deep that the squares of these distances are comparatively small. We will assume a splice as shown on Plate III. It is good practice to locate the splice at a stiffener.

The maximum moment the web can carry =

$$1/8\times16{,}000\times9/16\times100\times100=11{,}250{,}000 \text{ in.-lb.}$$

$$\begin{aligned}
1.5^2\times2&= 4.50\\
4.5^2\times3&= 60.75\\
7.5^2\times2&= 112.50\\
10.5^2\times3&= 330.75\\
13.5^2\times2&= 364.50\\
16.5^2\times3&= 816.75\\
19.5^2\times2&= 760.50\\
22.5^2\times3&= 1518.75\\
25.5^2\times2&= 1300.50\\
28.5^2\times3&= 2436.75\\
31.5^2\times2&= 1984.50\\
34.5^2\times3&= 3570.75\\
37.5^2\times2&= 2812.50\\
40.5^2\times3&= 4920.75\\
&\overline{20994.75}
\end{aligned}$$

The stress on the remotest rivet due to bending =

$$\frac{11250000 \times 40.5}{2 \times 20995} = 10{,}900 \text{ lb.}$$

The vertical component due to maximum shear is

$$\frac{327400}{70} = 4670 \text{ lb.}$$

The resultant of these is 11,850 lb.

One 7/8-in. rivet can carry in bearing on the 9/16-in. web $7/8 \times 9/16 \times 24{,}000 = 11{,}800$ lb. or in double shear 14,400 lb. The splice rivets as designed are sufficient and the splice may be located anywhere on the girder that may be found expedient.

The aggregate thickness of the two splice plates may be found by equating their section moduli to that of the web

$$1/6\ t \times 84 \times 84 = 1/6 \times 9/16 \times 100 \times 100$$

$t = 0.79$ in. or two 7/16-in. splice plates are required.

Splices of Flanges.—If necessary to splice the flange angles or flange plates it may be done by the method given in Chapter IV, page 115.

Total Dead Weight.—The dead weight of one girder is 36,980 lb. and the dead weight of one-half of the lateral system is 3290 lb., which gives a total dead weight of $36{,}980 + 3290 = 40{,}270$ lb. for one girder. This equals $\frac{40270}{70} = 575$ lb. per foot and checks the assumed dead load.

CHAPTER VI

BOX GIRDERS

General.—A design of a box girder of the form shown in Fig. 71, consisting of two webs with the flange angles and cover plates, is very simple, and in most of its features is similar to that of the ordinary plate girder. It quite often happens, however, in buildings, that the box girder must be built very shallow and be designed to carry a very heavy load at its center, generally from a column. These conditions render the use of three webs (Fig. 72) necessary in order to have sufficient material to rivet the

FIG. 71.—Two-web box girder.

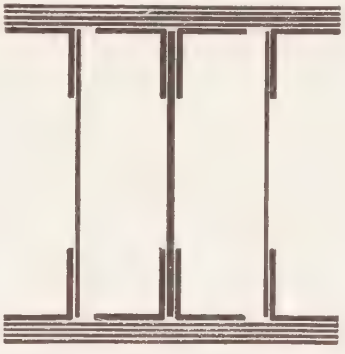


FIG. 72.—Three-web box girder.

flanges to, and give rise to many difficulties in the design. The center web is usually made double the thickness of the two outside webs, with two sets of flange angles, top and bottom. The outer webs are generally built with only one flange angle, owing to the impossibility of riveting inner flange angles to the cover plates, unless the girder is large enough for a man to crawl through. The thickness of the web will probably be limited by the necessary bearing strength of the rivets rather than from considerations of shear only. The rivets passing through the center web must each carry about twice as much stress to the flanges as those through the outer webs do and consequently must have double the strength in bearing. This is what leads to the making of the center web twice the thickness of the outer

webs. Considering this practical point, it is evident that any fine spun theories as to the actual distribution of the shearing stress between the three webs are useless.

A very heavy shear under such conditions, which is practically constant from the center to the ends, requires the closest possible spacing of rivets throughout in order to give sufficient strength. To accomplish this result, the neutral axis must be kept as close as possible to the center of the webs, vertically, in order that the rivet spacing may come out equal in both flanges. The small depth and heavy flanges make it necessary to apply the exact beam theory and to design in accordance with it. A preliminary design may be roughly made by applying the same theory that we apply to the ordinary deep plate girder. It must then be carefully revised and checked by means of the exact method, using the regular beam formula and computing the moment of inertia exactly. There is some difference of opinion as to the proper portion of the cross-section to consider in computing the position of the neutral axis and the value of the moment of inertia of the cross-section.

Location of the Neutral Axis.—The neutral axis evidently is in some definite location with reference to the cross-section. Its exact location is a matter of more or less uncertainty and is probably incapable of exact solution. It is possible, however, to find its probable extreme positions. Once these extreme possibilities are known the design can be made safe for the extremes. Then, whatever the true position is, the stresses will be within the assumed limits. To understand why its exact location is uncertain, consider for a moment whether the position of the neutral axis should be computed for the gross area of the cross-section or for the net area. If the net area obtained for the full length of the girder at every section, it would evidently be correct to compute the position of the neutral axis for the net area. The net area obtains, however, at a series of sections which are some little distance apart; and it varies gradually from a certain minimum amount up to the gross area as the plane cutting the section is moved along the beam. The gross area exists over the greater part of the length of the beam and consequently for the greater part of this length the neutral axes would pass through the center of gravity of the gross section. It happens, however, that in most cases the centroids of the gross and net areas are nearly coincident. Another element which we have not yet mentioned

enters into the situation and complicates it. It is usual to consider that on the compression side the rivets fill the holes and can take compression. In consequence of this, when designing by the approximate method, the gross area is taken to be effective on the compression side, whereas the net area is used on the tension side. It is doubtful, however, whether the position of the neutral axis should be computed on this basis, although this element should be understood to affect the result in some degree. A beam when loaded bends in a smooth curve which would lead us to the conclusion that the neutral surface, which is composed of the successive neutral axes, also forms a smooth curve. If this be the case, as seems entirely reasonable, the neutral axes for all sections will remain at practically a constant distance from top or bottom of the beam. They will probably also stay nearer to the position occupied in the predominating section, which is the gross. The various axes occupy some compromise position and certainly do not jump up and down whenever a rivet hole is encountered. The extreme positions can be found by computing for the gross area, the net area, and the gross area on the compression side in combination with the net area on the tension side. Generally, it is sufficient to take the neutral axis as passing through the center of gravity of the gross area of any section, but all the possibilities should be considered in careful designing. As stated before, if the section be made of sufficient strength for the extreme positions of the neutral axis, it will evidently not be overstressed, whatever the position of the neutral axis may be.

Computation of Moment of Inertia.—The moment of inertia may be computed for either of the three conditions outlined in the preceding paragraph. Whether the gross area, the net area, or a mixture of the two will be used depends to some extent upon the result which is sought. For instance, when computing deflections the moment of inertia of the gross area should be used because the gross area obtains over such a large proportion of the length of the beam. Many, perhaps the majority, of engineers claim that the moment of inertia of the net section should be used in designing. This method has the advantage of simplicity and is on the safe side. It generally leads to the use of a slight excess of material, but simplifies many matters connected with the design to a considerable degree. There are some arguments in favor of using the gross section on the compression side and the

net section on the tension side when designing. This method is entirely consistent with the common theory of design of plate girders as well as of trusses and gives a somewhat more economical design than when the moment of inertia of the net area is used. It leads, however, to very laborious and involved computations in which there is a considerable liability to error. A value determined by averaging the moments of inertia of the gross and net sections is advocated by some designers. This method has the advantage of simplicity and, considering the many uncertainties involved, is sufficiently accurate for all practical purposes.

Computation of Pitch of Flange Rivets. The proper method of computing flange rivets in box girders is by the exact formula $S = \frac{VQ}{I}$. It should be noted that this formula is that for finding the horizontal shear per inch of length at a given section, and consequently it is to some extent making an assumption to apply it to the shear on a vertical surface of a rivet connecting different parts. This assumption is, however, entirely reasonable, and can be trusted to give results which are close to the truth. Another method which can be used wherever the shear is constant between two points is to find the difference in flange stress between the two points and provide a sufficient number of rivets in this distance to transfer the difference in flange stress from the flange to the web. These two methods should check when accurately applied to a given case. The point in the flange where the rivets are located has some influence upon the way in which they carry their stress. For instance, assuming a case where the shear is constant for some distance, and applying the second of the two methods stated above, it is evident that the rivets which take stress from the flange should be located as nearly as practicable in the same horizontal plane as the center of gravity of the flange. If located some distance from this plane an eccentricity exists which has the effect of overloading the rivets. This can be avoided in practical designing by not using too wide an angle leg against the web. It is possible to conceive of a case where the widening of an angle leg to accommodate more flange rivets would actually produce a greater stress on the rivets through eccentricity than a smaller number of rivets would have been called upon to carry had they been located nearer to the center of gravity of the flange.

Disposition of Flange Area.—The rivets are assumed to carry a certain amount of stress from the webs to the flanges, and care must be taken to see that the webs and flange angles are so located that the proper amount of cover plate area is tributary to them. For instance, in Fig. 73, when planes *v–v* are passed vertically through the girder at points equidistant (*k*) from outer and inner rows of rivets, each part of the girder so divided should be capable of standing alone as a girder and of carrying its portion of the total load without overstressing any of its parts. Economy also dictates that no part should be understressed.

It may not be possible to *exactly* fulfil these conditions, but the designer should adhere to them as closely as possible.

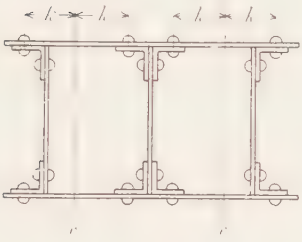


Fig. 73.

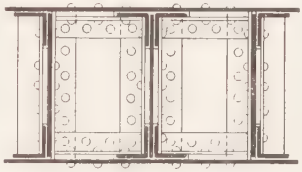


Fig. 74

Diaphragms.—In order to insure that all similarly located portions of any cross-section of the girder shall be under similar conditions of stress, the various webs should be connected by diaphragms attached to the web stiffeners at intervals depending upon the manner of loading and also upon the judgment of the designer (see Fig. 74). Diaphragms should be used at all points of concentrated loading as well as at reactions. Sometimes conditions of loading and construction are such that it is impossible to rivet such diaphragms in place. It is better to bolt them in place than to omit them. Where neither bolting nor riveting is practicable it may be necessary to omit them entirely at some points. As their function is to distribute stress and as it will be largely accomplished if the vertical deflections of the three webs be made constant, the diaphragms may, if necessary, be riveted to the center web only, closely fitted against the cover plates both top and bottom, and extended out against the outer web. A detail of this kind is somewhat faulty, as it transfers vertical stress through the cover plates when it should be carried directly into the web.

It is, however, better than none. The judgment of the designer must be relied upon to fix the sizes and riveting of the various parts of such diaphragms.

Design of Typical Three-Webbed Box Girder.—The design of a three-web box girder for certain assumed conditions will be given in order to illustrate the various problems encountered and the results which may be expected.

Type.—Three-web box girder.

Span.—19 ft. center to center supports.

Loading.—445,000 lb. concentrated load at center. 10,000 lb. is the assumed weight of girder and is assumed to act as a concentrated load at center. The weight of the girder is really a distributed load of greater intensity at the center than at the ends. The assumption that it is concentrated and acts at the center is then on the safe side.

Moment.—WL/4 giving 25,935,000 in.-lb. (max.)

Shear.—227,500 lb. (maximum).

Max. fibre stress in tension and compression, 15,000 lb. per square inch.

Max. fibre stress in shear 12,000 lb. per square inch on net area of webs.

Max. fibre stress in shear 10,000 lb. per square inch on rivets.

Max. fibre stress in bearing 18,000 lb. per square inch on rivets.

Girder must not occupy more than 30 in. in depth over all nor more than 28 in. in width at any point.

We will first make a tentative design by the approximate theory.

$$\text{Webs, required net area} = \frac{227500}{12000} = 19 \text{ sq. in.}$$

We will assume that the webs will be 24 in. deep, and will also make the usual assumption that the net area is three-quarters of the gross.

$$\text{Total thickness} = \frac{19}{24 \times 3/4} = 1.06$$

This will give approximately one 5/8-in. and two 5/16-in. webs.

We will further assume that the effective depth is equal to the depth of the web.

$$\text{Flange stress} = \frac{25{,}935{,}000}{24} = 1{,}080{,}600 \text{ lb.}$$

	Top flange		Bottom flange
Required area $\frac{1080600}{15000}$ =	72.04		72.04
Web equivalent 1/8×5/4×24 =	3.75		3.75
	68.29		68.29
Four-6×6×1/2-in. L's	23.00		19.00
	28)45.29		24)49.29
Total thickness cover plates =	1.62		2.06
Four-7/16-in. plates =	1.75	1-9/16 in. plates	0.56
		3-1/2 in. plates	1.50
			2.06

The distance back to back of flange angles we will take as 24-1/2 in. to allow for irregularities in the edges of the webs. We will design first by using the moment of inertia of the net section. As it is desirable, for reasons already given, when using this method, to make the top and bottom flanges alike, we will assume four 1/2-in. cover plates.

Cutting off of Cover Plates. For the same reasons outlined on page 111 corresponding top and bottom cover plates should be dispensed with at the same distance from the center. This keeps the neutral axis in practically the same horizontal plane and avoids secondary stresses which are caused when the position of the neutral axis is suddenly varied in a vertical direction.

Disposition of Material in Flanges. The disposition of material in the flanges is also very important because of its effect upon the position of the neutral axis. The proper way to arrange the material will depend to some extent upon the assumptions made and upon the method of procedure in designing. For instance, when designing by using the moment of inertia of the net cross-section, both flanges should be of the same composition. This gives a horizontal axis of symmetry which will be the neutral axis for both gross and net sections. As the same tensile and compressive fibre stresses are commonly used when designing according to this method, a symmetrical section will be the most economical as well as the easiest to design.

When designing using the gross section in the parts of the beam subjected to compressive stresses, and the net section in the parts subjected to tensile stresses, a "balanced" or symmetrical section is preferable if the centroid of the section is assumed to be that of the gross area. If the centroid is assumed to be that found by considering the gross area above the neutral axis and the net

area below it, the lower flange should be made heavier than the upper in order to keep the centroid as close as possible to the mid-depth of the webs. The object in this is to keep the statical moments of the top and bottom flanges as nearly equal as practicable in order that the same rivet spacing may be used in each one. This is not done for the primary object of using the same rivet spacing, but because it is quite often necessary to use the minimum spacing in order to obtain a sufficient number of rivets. The arrangement of material should evidently in such cases be such as will not call for a closer spacing in one flange than is needed in the other.

The next step is to compute the moment of inertia of this section. The tables in the back of the book give a ready means of quickly computing the moment of inertia of any combination of webs, angles, and cover plates. We will, however, go through with the computation in order to illustrate the methods and arrangement of computations of this character. By arranging the work as shown below and computing first the moment of inertia of the gross section and then subtracting the moment of inertia of the rivets we may readily obtain the moments of inertia of both the gross and net sections with a minimum of computation. The first column gives the portion of the section used; the second column its area; the third column the distance (d) of the center of gravity of this area from the assumed axis (for convenience generally taken at the mid-depth of the web), the quantity (d) has sign and is plus or minus according to whether it is above or below the axis; the fourth column is the product of the second and third, the fifth column is the product of the third and fourth, and the sixth column is the moment of inertia of each of the parts about its own gravity axis. Adding the fourth column with due attention to sign evidently gives the moment of the area about the assumed axis. Dividing this sum by the total area gives the distance from the assumed axis to the gravity axis. A minus sign indicates that the axis is on the minus side of the assumed axis. The sum of the last two columns gives the moment of inertia about the assumed axis. In cases where the assumed axis is found to be different from the gravity axis, if the product of the area of the cross section by the square of the distance between the assumed axis and the gravity axis be subtracted from the sum of the last two columns, the result will be the moment of inertia of the section about its gravity axis. The quantity to be subtracted is most easily found by multiplying

together the two quantities which will occur in such cases at the foot of the fourth column.

If the radius of gyration is desired its square may be readily found by dividing the moment of inertia by the area. After finding the moment of inertia of the gross section, we will find that of the net section. This is most readily done by adhering to our original axis and finding the moment of inertia of all the rivet holes about it. The moment of inertia of the holes

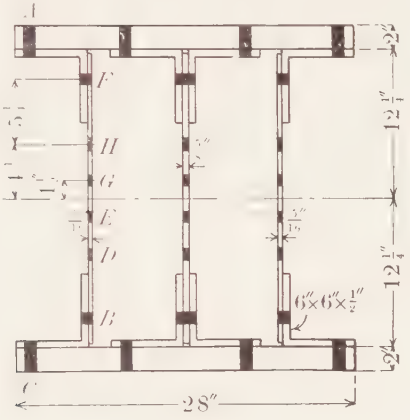


FIG. 75.

about their own gravity axes may be neglected. As we have assumed 7/8-in. rivets, the holes will be taken as 1 in. in diameter. The holes in the web are assumed as shown in Fig. 75 and will occur at the point of maximum moment as it will be necessary to use stiffeners under the column which rests upon the girder in order to transfer the load to the web. The areas of the holes are all negative as they are taken away from the section. The sign $\pm$ before d indicates that the distances are equally positive and negative, hence Ad will be O. Ad^2, however, will be of the same sign as A.

Part	Area = A	d	Ad	Ad^2	I_g
3 webs	30.00	0	.....	.....	1440
4 top L's	23.00	+10.57	+243.1	2,570	80
4 bottom L's	23.00	−10.57	−243.1	2,570	80
4-28×1/2-in. pls.	56.00	+13.25	+742.0	9,832	18
4-28×1/2-in. pls.	56.00	−13.25	−742.0	9,832	18
	188.00	.....	.....	24,804	1636
				1,636	
				26,430 = I gross area	

Rivet holes	A	−10.00	+13	−130	− 1,690
	B and F	− 6.50	± 9.75	0	− 618
	C	−10.00	−13	+130	− 1,690
	D and H	− 2.50	± 4.5	0	− 51
	E and G	− 2.50	± 1.5	0	− 5
		31.50			4,054

22,376 = I net area

The maximum fiber stresses will be

$$f_c = f_t = \frac{25{,}935{,}000 \times 14.25}{22{,}376} = 16{,}520 \text{ lb. per square inch}$$

As this is more than 15,000 we will need to revise our section by adding some material. A fairly close estimate of the amount of material that will need to be added may be made by computing the moment which the cross-section already designed will bear at a stress of 15,000 lb. per square inch on the remotest fiber. The moment thus found should be subtracted from the moment the girder must carry. The difference in moment must be carried by the additional thickness of cover plates. This difference in moment should be divided by the depth out to out of cover plates of the section already designed. This quotient should then be divided by the allowable fiber stress. The result will be very nearly the required area.

Moment section will carry is $\frac{22376 \times 15000}{14.25} = 23{,}550{,}000$

$25{,}935{,}000 - 23{,}550{,}000 = 2{,}385{,}000$

Required area $= \frac{2385000}{28.5 \times 15000} = 5.58$ sq. in. net

Dividing the required area by the net width gives the additional thickness required. $\frac{5.58}{24} = 0.2325$ in. or 1/4 in. Four 28 in. × 9/16 in. plates should be used in place of the four 28 in. × 1/2 in. plates assumed.

The computation of the moment of inertia of the net section is as follows. The letters following the rivets refer to rivets corresponding in position to those in Fig. 75.

Part	Area	d	Ad	Ad^2	I_g
3 webs	+ 30.00	+ 0	0	0	1440
4 top L's	+ 23.00	+10.57	+243.1	2570	80
4 bottom L's	+ 23.00	−10.57	−243.1	2570	80
4-28×$\frac{9}{16}$ top	+ 63.00	+13.37	+842.3	11260	27
4-28×$\frac{9}{16}$ bot.	+ 63.00	−13.37	−842.3	11260	27
Rivets A	− 11.00	+13.12	−144.3	− 1893	
Rivets C	− 11.00	−13.12	+144.3	− 1893	
Rivets B	− 3.25	− 9.75	+ 31.7	− 309	
Rivets F	− 3.25	+ 9.75	− 31.7	− 309	
Rivets D	− 1.25	− 4.50	+ 5.6	− 25	
Rivets B	− 1.25	+ 4.50	− 5.6	− 25	
Rivets E	− 1.25	− 1.50	+ 1.9	− 3	
Rivets G	− 1.25	+ 1.50	− 1.9	− 3	
	168.50			23,200	1654
				1,654	
				24,854	

$$f_c = f_t = \frac{25935000 \times 14.5}{24854} = 15{,}130 \text{ lb. per square inch.}$$

This stress is about 1 per cent. in excess of that allowed. The stress we are using is conservative and the using of the net area in designing is probably somewhat on the safe side. As the author's object is to illustrate principles, and not merely to show how to conform to specifications, we will make no further revision of the section. If such an excess occurred under similar circumstances in practice, the author would allow it without question. In order to show what difference other methods of designing will make, we will find the maximum stresses in the beam designed, making the following assumptions. The moment of inertia will be computed for the gross area on the compressive and the net area on the tensile parts of the section. The neutral axis will be located both for gross section and for the combination of gross and net section given above. The additional computations required are made below.

The position of the center of gravity is found by dividing the algebraic sum of the moments Ad by the total area A. In this case it is $183.5 \div 185.25 = 0.99$. As the sign is plus the gravity axis is 0.99 in. above the center of the web. To find the moment of inertia about this axis, subtract the total area of the section multiplied by the square of the distance between the two axes from the value of I already obtained. This will be seen to equal the product of the two quantities, 183.5 and 0.99, at the bottom of the fourth column.

Part	Area	d	Ad	Ad^2	I_g
Gross area	202.0	0	0	0	29,314
Rivets C	− 11.00	−13.12	+144.3	−1893	
Rivets B	− 3.25	− 9.75	+ 31.7	− 309	
Rivets D	− 1.25	− 4.50	+ 5.6	− 25	
Rivets E	− 1.25	− 1.5	+ 1.9	− 3	
	+185.25		+183.5	−2230	29,314
			+0.99		−2,230
					27,084 = I of section about centroid of gross area.
					26,902 = I of section about centroid found by taking out rivet holes in tension side of neutral axis.

We will now find the maximum tensile and compressive stresses for these two cases. In both of them the net area has been used on the tension side and the gross area on the compression side of the neutral axis. The neutral axis in the first case is that of the gross section and in the second is that of the combination gross and net section.

In the first case

$$f_t = f_c = \frac{25935000 \times 14.5}{27084} = 13{,}910 \text{ lb. per square inch.}$$

In the second case

$$f_c = \frac{25935000 \times 13.51}{26902} = 13{,}050 \text{ lb. per square inch.}$$

$$f_t = \frac{25935000 \times 15.49}{26902} = 16{,}040 \text{ lb. per square inch.}$$

For convenience of comparison the results of the three cases are tabulated below.

Location of neutral axis	Moment of inertia taken for	Value of moment of inertia in^4	Distance to remotest fiber		Fiber stresses remotest fiber lb. per sq. in.	
			Comp.	Ten.	Comp.	Ten.
Net section.	Net section.	24,854	14.5	14.5	15,130	15,130
Gross section above net section below.	Gross section above and net section below neutral axis.	26,902	13.51	15.49	13,050	16,040
Gross section.	Gross section above and net section below neutral axis.	27,084	14.5	14.5	13,910	13,910

Computation of Flange Rivet Pitch. -The actual computation of the flange rivet pitch in the first two cases is very simple. Most of the required quantities can be taken directly from the tables made in connection with the computation of the moment of inertia. If the webs are properly spaced so that a proportional amount of flange area is tributary to each one, the horizontal shear per inch of length of girder may be computed for the whole flange and then divided among the different connections to the webs in proportion to their thickness. A little reflection will show that in a properly designed three-web girder the flange rivet pitch should be the same in all three webs.

When the moment of inertia of the net section is the basis of the design and the two flanges are alike, the rivet pitch will be the same in both flanges. We will compute the horizontal shear per linear inch by using the statical moment of the net section of the flange. As we are using the moment of inertia of the net section it seems fairer to use the net section throughout.

The computation of the statical moment can readily be made from the table on page 153. It is as follows:

$$+243.1+842.3-144.3-2\times 9.75=921.6$$

The quantity (2×9.75) is the moment of the holes B in the angles only, about the neutral axis.
The vertical shear equals 227,500 lb.

$$\text{Horizontal shear}=\frac{227500\times 921.6}{24854}=8435 \text{ lb. per lineal inch of}$$

girder. This should be divided by 4 to get shear on one outer web. This gives 2109 lb. The value of one rivet is either $5/16\times 7/8\times 18{,}000=4922$ lb. or $0.6\times 10{,}000=6000$ lb. The rivet spacing must equal $\frac{4922}{2109}=2.334$ or 2-5/16 in. in two lines.

If the moment of inertia and statical moment of the gross areas be used the computation will be as follows:

$$\text{Horizontal shear}=\frac{227500\times 1085.4}{29314}=8424 \text{ lb. per lineal inch.}$$

$$\text{Rivet spacing}=\frac{4922}{2106}=2.337 \text{ or 2-5/16 in. as before.}$$

It is less work and gives practically the same result to use the properties of the gross areas in computing the rivet pitch.

In the second case, taking the combination of gross and net

areas and using the centroid of the gross section, the compression flange will limit the rivet pitch.

The statical moment of the compression flange about the neutral axis will be 1085.4

$$\text{Horizontal shear} = \frac{227500 \times 1085.4}{27084} = 9117 \text{ lb.}$$

Dividing the shear by 4 as before

$$\text{Rivet pitch} = \frac{4922}{2279} = 2.159 \text{ or } 2\text{-}1/8 \text{ in.}$$

In the third case, taking the combination of gross and net areas and using the centroid of the combination of areas, we must compute the pitch for both flanges.

The statical moment of the top flange is found as follows:

Part	Area	Dist.	Ad.
4 top plates	+63	+12.38	+ 780
4 angles	+23	+ 9.58	+ 220
			1000

The statical moment of the bottom flange is found as follows:

Part	Area	Dist.	Ad.
4 bottom plates	+63	−14.36	−904.8
4 bottom angles	+23	−11.56	−265.9
Rivets *C*	−11	−14.11	+155.2
Rivets *B* (partial)	− 2	−10.74	+ 21.5
	+73		−994

The rivet pitch for the top flange is found as before

$$\text{Horizontal shear} = \frac{227500 \times 1000}{26902} = 8470$$

Dividing the shear by 4 as before

$$\text{Rivet pitch} = \frac{4922}{2118} = 2.32 \text{ or } 2\text{-}5/16 \text{ in.}$$

The rivet pitch for the bottom flange is found similarly

$$\text{Horizontal shear} = \frac{227500 \times 994}{26902} = 8420$$

Dividing the shear by 4 as before

$$\text{Rivet pitch} = \frac{4922}{2105} = 2.33 \text{ or } 2\text{-}5/16 \text{ in.}$$

In this case the flange rivet spacing is practically constant whatever assumptions are made in the design.

Stiffeners at Points of Concentrated Loading. Stiffeners will be put on all three webs at points of concentrated loading. One-fourth the load would be assumed to come on each of the outer webs and one-half on the center web. The design of the stiffeners and the rivet spacing in them is similar to the design of the end or reaction stiffener in the deck plate girder (see page 134). (See also paragraph on diaphragms page 147.)

Cutting off of Cover Plates.—This may be done either analytically or by diagram and the method is similar to those explained elsewhere (see pages 109 and 136). The area required in the flanges in this case will vary directly as the distance from the reaction because the load is concentrated.

CHAPTER VII

SHOP HINTS FOR STRUCTURAL DRAFTSMEN

By John C. Moses, M. Am. Soc. C. E.

1. THE DRAFTSMAN AND THE TEMPLET MAKER

General.—The manufacture of structural steel work may be divided into the four operations of drafting, templet making, shop work and erection. The draftsman's portion consists in making drawings in accordance with the designs of the engineer for the guidance of the workmen in the remaining operations. As the engineer's designs are frequently incomplete it may also be part of the draftsman's duties to design details of joints, rivet spacing, etc. This part of his work is described elsewhere. In this chapter we shall consider his work as part of the process of manufacture after the design is complete.

This subject is of growing importance. Drafting offices at the present day are not in as close touch with the shops as formerly, and most draftsmen have little personal knowledge of shop operations. As a result drawings frequently call for details that could easily have been so modified as to reduce shop costs; but when these drawings reach the shop they have been checked and approved and it is a troublesome matter to make any changes in them. The badly placed rivets are driven by hand, two sets of templets are made where one might have done, the erector cuts off his gussets to get in his bracing, etc. Each one has his opinion of the draftsman, but the culprit is not there to see or hear, and the matter is not important enough to lead a busy foreman to enter a formal complaint. The draftsman keeps on copying the same detail for subsequent jobs, and in the aggregate the increase in cost is considerable.

Further, the cost of drawings is to-day a larger part of the total cost of manufacture than it used to be. The draftsman must justify this by making possible a decrease in the other shop operations sufficient to make a reduction in the total cost of manufacture. This requires an understanding of the operations subsequent to his own.

This chapter is written in the hope that it will be useful not only to the draftsmen employed in structural shops, but also to those elsewhere employed, for economy of production is one of the essentials of good engineering.

Templet Shop.[1]—We will consider first the templet shop and see what the draftsman can do to save expense in that department while still conforming to the engineer's requirements. He should realize first of all that the cost of the templet department is about two-thirds that of the drafting department, and that the two combined may spend as much on a job as is spent in the actual construction in the shop. Therefore any reduction of expense in the templet shop is decidedly worth while.

It takes about as much time to make the templets for a job as it does to make the drawings. The drafting department, however, generally uses up most of the available time and so the templet foreman must put a number of men on the job at once. Therefore each drawing should be made complete in itself. If two or more sheets can best be laid out by one templet-maker a note on each sheet should say so. The shop bills should be separate for each sheet. General notes giving size of rivets, reaming directions, etc., should appear on each sheet. Identical pieces appearing on different sheets may be given the same name and be referred to on all the corresponding shop bills but one as "old templet, page —"; when so noted they are passed over by the templet maker and are made by the man having the page not so marked. But all the dimensions must be given on each sheet to enable the connecting parts to be made without looking up any other drawing. When the templets are checked such a one is tried in each place it is to go and the total number wanted placed upon it.

But if the job is put into the shop piecemeal, so that punching will begin before the templets are done, it is often best to name the pieces on each drawing as if they did not appear elsewhere; it will be easier to make the templet over again than to get back the former one and check the new work with it. This case frequently happens in building work. Pieces should not be named alike

[1] A templet is a wooden pattern made in the shape of the steel piece to be made, with holes drilled in it to match the rivet holes, etc., in the steel piece. It is clamped to the steel piece, the holes are marked by a center punch, and lines are drawn on the steel to correspond to the edges of the templet and to mark the edges of the piece.

when this is done, for templet checkers pass a shop-driven stiffener, for instance, if it matches its girder web, and do not try the actual spacing. The templet maker lays out a spacing once and then transfers the marks to the connecting pieces, and so may have them match, although not like the drawing. If the piece appears on another drawing without being noted as an "old templet" it may be made a little differently the second time and not detected. In the shop the stiffeners that match one girder may be put on another web, with poor results.

When parts cannot be made identical it is desirable to draw them so that several can be placed on one templet. This saves lumber, which is expensive, and takes time to prepare; it saves time in making the templets, and it makes less templets to handle in the shop or to store. The similar pieces should be drawn on one sheet; frequently they are represented by one drawing, with notes corresponding to those the templet-maker will use on his templet. In the latter case, do not note a detail as "Omitted on Col. 3 only," but note it thus: "On Cols. 1, 2, 4, and 5." The templet-maker will surround the holes on his templet with a ring containing the numbers 1, 2, 4, 5, and he wants *positive* orders, not negative ones, on his drawing. If similar pieces differ in length by a few inches, throw all the difference into one end, if possible, or else into the two ends, but not into the middle. Then most of the templet holes will be alike for both kinds of pieces. Arrange the holes that are not alike so that they will be at least 3/8 in. apart on the templet; then they can be bored without splitting out the wood between the holes.

Make rivets in girder flanges opposite, and not staggered, if the engineer will allow it. The theoretical advantage of staggering rivets in the two legs of wide angles is, to say the least, problematical, while there is a distinct advantage in placing the rivet on the wide gage[1] of one leg opposite that on the narrow gage of the other leg, since in this case one hole in the templet marks both rivet holes. It should be said, however, that where angles are thick or gages narrow, staggering the rivets makes it easier to drive them; this point is considered much more important in some shops than in others.

In making templets for plate girders a board is first laid off to represent one leg of a flange angle. All odd holes in either leg

[1] The "gage" or "gage-line" is the longitudinal line upon which rivets are placed.

of any flange angle are included. Four similar boards are clamped under this one, and then all five are bored at once. Two of them are battened together for the cover plate pattern, and the other two form the top and bottom of the web-plate pattern. A stiffener or web-plate splice is laid out similarly, and with it are bored pieces for the web templet. In order to make this method of templet-making applicable to cambered girders, the camber[1] is produced by making one corner of each web section slightly less than a right angle, keeping top and bottom flanges alike.

For beam work use standard details as far as possible. Boards just large enough to contain the group of holes for each standard connection are kept at the marker's bench, and the beam templet consists of a pole with center lines and names of connections on it. This pole is laid on the beam, and the connection templets are moved to the proper mark and set at the proper height by gages that hook over the top of the beam and drop into holes in the boards.

Pole templets are strips of wood 1-1/4 in. wide by 1/2 in. thick, planed on all four sides, and are of any desired length. They are also used for angles having one gage-line on each leg, the holes being located by lines drawn across the pole. One side of the pole gives the holes in one leg of the angle and the other side gives the holes for the other leg. When marking the steel the pole is placed alongside the angle, or between two angles, if a right and left pair are wanted, and lines drawn square across the leg of the angle by means of a small try-square. The gage of the holes is determined by a gage on the punching machine. Pole templets are quickly made, contain little lumber, and can be used several times by planing off the old marks.

Webs of similar truss posts or diagonals can often be made alike when their flanges are different. Templets for such webs are made of two narrow boards battened together. If alike except for transverse distance of holes apart, the boards can be bored from one lay-out and set the proper distance apart when battened together. A small and a large gusset can be put on one templet if the holes in one will match the holes in the other as far as they

[1] Camber is a slight vertical curve put into bridges. It is usually made of such an amount that the bridge will be horizontal when fully loaded. It is used partly for appearance as a truly horizontal line will appear to the eye to sag.

go. It is sometimes advisable to vary the gages in angles from the standard in order to make tie plates and lattice bars alike.

In planning pieces to go on the same templet, it should be remembered that it is not usually best to make templets with detachable ends to be changed for different pieces, or to make them in half lengths to turn over. Templets are made of narrow boards battened together and cannot be turned over to advantage, and detachable parts are apt to be inaccurately matched to the main templet when put together in the shop for the purpose of marking the steel.

The dimensions on drawings are primarily for the templet-maker's use. First of all, he will want to set his helper to work to get out the lumber. For that purpose the drawings should state the length and width of all gussets and lengths of all other pieces. Web splices of girders should be located on a separate dimension line. Locate all web stiffeners similarly, also ends of flange plates. Tie in rivet spacing with other dimension lines at every opportunity. A templet-maker lays out rivet spacing by stretching a tape, usually divided to eighths, beside his board. If he comes out wrong at the end of a long line he has to go all over it again, erasing his old marks; it will save him much time to have his spacing tied in at every stiffener, web splice and flange plate end. He works with a soft pencil on soft wood, and sixteenths make pretty fine work, to be avoided as much as possible. When a number of equal spaces are used, always give the total they make up. Unfortunately, templet-makers can rarely be brought to see the use of doing anything twice by different methods as a check; rivet spacing on drawings should therefore be checked with general dimensions.

The shop bills should be carefully checked in the drafting-room, for a templet-maker may copy the number of pieces there called for, regardless of the drawing. Write "exact" after dimensions that must not be exceeded. Give the total distance between holes in the outstanding legs of end connection angles, but also give gages in every case for the templet-maker. Note when uprights, fillers or splice plates must be fitted to flange angles. Avoid double cuts on one leg of an angle, as they involve making a special cutting pattern to go with the templet pole.

When showing only part of a given piece, always draw the *left-hand end* of the piece, as the templet-maker must work from left to right with his tape. Use uniform methods, putting general

notes in the same part of all drawings, and writing sizes of stock in the same way every time, and on the left ends of the pieces. Do not change existing customs without exceptionally good reasons. Uniformity of drawings not only saves much time when looking for information in the shop, but also much liability of mistake from oversight. In general, remember that the templet-maker has to redraw everything and should have dimensions given so as to make it easy to do this.

There are many advantages in laying out work full size on a floor in the templet shop. Truss members and connecting gussets are then checked as to length of pieces and matching of holes, and interferences are detected. When this is customary the drawings can be simplified to great advantage and at the same time lessen the work of the templet-maker, who is no longer obliged to follow exact spacings. Many connections can be laid out by him as well as by the draftsman, thus saving the draftsman's time. A disadvantage of this method is the impossibility of dividing the templet work among as many men; another is the lack of the kind of men formerly found in templet shops.

Pattern Making.—Most of this work will consist of shoe plates, or machinery for draw-bridges. This work is expensive and can often be simplified by making patterns that are altered to make the different castings by means of detachable pieces. Old patterns can often be used again, with perhaps some changes. As usually but one or two pieces of a kind are wanted, it is often cheaper to use an old pattern that is heavier than needed, the cost of a new pattern being more than the value of the iron saved. Cores are expensive to make, and, as shoe plates are designed for bearing values of masonry, the top surface can be ribbed to save metal instead of coring out the inside. Straight surfaces are much cheaper than curved ones, which latter must often be whittled out by hand.

The cost of the pattern rather than the amount of iron is usually the determining factor when there are but few pieces of a kind. The reverse is true when there are many of a kind, and a little calculation of weights and costs will often determine the best thing to do.

Make casting drawings correctly to scale, and to a larger scale than is usually used for the other drawings. Draw them on separate sheets, as they go to the pattern-maker, while the other drawings go to the templet-maker. Note if holes are to be cored

or drilled. If the latter, a templet will be made by which to lay out the holes, as on other pieces of iron. Cored holes are likely to be irregular in size and incorrectly located, and should not be used for machinery connections. Always note all surfaces that require planing, as an additional thickness of pattern is there required. A pattern-maker can do almost anything when necessary, but very slight modifications in design will frequently cut the cost in half.

II. The Draftsman and the Bridge-Shop

General.—In the previous section attention was directed to templet making, and some of the ways were pointed out in which the draftsman could decrease the expense of that part of the process of manufacturing structural work. After the templets are made the steel must be cut, punched, assembled and riveted. In this work, also, it is true that the draftsman can often reduce costs by understanding the methods of the shop and planning the work with these methods in view. Two ways that are equally good from a theoretical point of view may differ widely in point of economy; even where one way is theoretically or aesthetically better than another, the advantage may be gained at a cost that is much greater than the designer would consider himself justified in paying if he knew of it. And sometimes carrying out a theoretically better plan necessitates a kind of shop work that is not as reliable as are the ordinary methods, and the final result is a poorer instead of a better product. The following notes on the principal operations in the shop will show some of the considerations that should govern the draftsman in his work.

Shearing Angles—Angles are cut to length and to the required bevels at the ends on a special machine having a knife with two cutting edges, one horizontal and one vertical, as shown in diagram by Fig. 76. This knife can cut both legs of an angle at the same time. The whole machine stands on a turntable, and the angles to be cut rest on horizontal skids. Square cuts are made by placing the machine at right angles to the stock on the skids. In this case the templet need only indicate the length, and the marker draws a square line on the angle with his try-square. A permanent guide line on the machine shows when it is set square to the work. To make bevel cuts, the machine is swung to the bevel given by the templet, or the angles themselves are swung around

when not too long or heavy. If a "front cut" is to be made (see Fig. 77) the angle is supported by the table of the machine, as in square cuts; but if a "back cut" is wanted the angle must be put

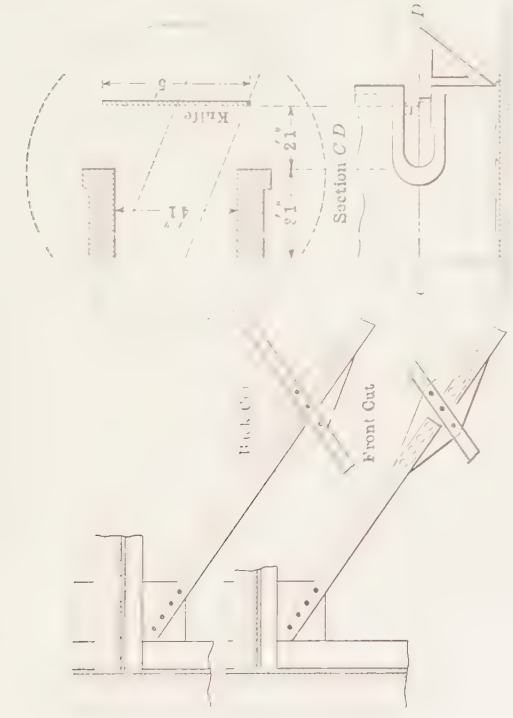


Fig. 76.

Fig. 78.

Fig. 77.

Fig. 76.—Capacity diagram of an 8 × 8 in. angle shear, showing factors that determine limiting bevel cuts.

Fig. 77.—Two different ways of cutting ends of bracing angles.

Fig. 78.—Capacity diagram of a 48 in. plate shear, showing factors that determine limiting bevel cuts.

into the other side of the knives, on account of the housing or frame of the machine, and is generally carried around to the back side instead of turning the machine entirely around. In either

case the degree of the bevel is limited by the construction of the machine, as an examination of Fig. 76 will show. For front cuts this bevel varies with the width of the leg cut, and a record of these limits should be in the drafting office. The limiting bevel for back cuts is the same for all sizes.

The bevel cut is made on the horizontal leg of the angle and the vertical leg is left square. If the bevel is a front cut, the angle can be turned over and the other leg also be given a front cut, but one leg cannot have a back cut and the other a front cut, nor can both legs have back cuts.

If bevels outside the limits are necessary, the angle is cut square and then the beveled leg is cut on the plate shear, or else is punched off. This leaves the square leg with a square face. Bevels inside the limit, on the other hand, should be shown as cutting the square leg to a sloping face to save a second cut on the machine.

Fig. 77 shows two ways of drawing an angle for a lateral brace or truss member; the lower sketch shows the angle with square ends, the upper one the same angle with bevel ends. Where the ends are beveled the operations in cutting the angle in the shop are as follows: A cutting templet must be made for each end in addition to the regular pole, thus doubling the templet work. The angle is first cut square to length, then carried around to the rear of the machine and the back cut made; then to the front of the machine and one front cut made; then turned end for end and the other beveled cut made. If the angle is long or heavy, the machine must be swung for each of these different cuts. Care must also be taken to cut the bevels the correct hand. The economy of cutting angles square will be evident, even if the gusset has to be made larger. The effect on the appearance of the work is much more apparent on the drawing than on the work itself, since the angle leg is flat against a large plate of the same color after painting, and is so much less conspicuous than the outstanding leg that it will not be noticed. In this case, as in many others, the draftsman should frequently examine finished work in order to have a true conception of the result of his drawing.

Shearing Plates—Small gussets are generally cut from large pieces on a shear arranged as shown by Fig. 78. Frequently a number of them can be made with two sides parallel, of the same width; they can then be cut from long plates of that width with few

cuts and little waste. When too irregular in shape for this, they should be planned with as few sides as possible, as every side represents a cut of the shear. The size of plate from which they are to be cut should be kept in mind and the gussets planned to use up the material with as little waste as possible. Corners much less than 90° should be avoided, as the plate will curl when sheared and not fit snugly to the adjoining parts.

Large gussets may be ordered from the mills cut to the shape desired. They are then known as sketch plates, and an extra charge of one-tenth cent a pound is made for them. It is a good plan to figure their cost and compare it with the cost of a rectangular plate of sufficient size, crediting to the latter cost one-half the value of the plate cut off as good material for future work, and the other half at scrap value. The rectangular plate will often be found the cheaper. Plates ordered "sketch" from the mills generally have to be trimmed again on the shears in any case, and are quite often cut wrong, or delayed in shipment.

An examination of Fig. 78 will show that long plates to be cut in two on a bevel are limited in position by the frame of the shear. The wider the plate the less the possible deviation from a square cut. This must be kept in mind in ordering stock, and a diagram of the shop shear should be among the draftsman's data.

Reëntrant angles on gussets should be avoided, as they cannot be cut by the shear, but must be punched out. The shear will not cut off a strip of less width than about half the thickness of the metal, and there is, of course, a limit to the thickness it is safe for any given machine to cut.

Cutting Beam Work.—Beams, channels, zees and tees are generally cut in pieces by special shears or by a saw. Shears cut them off square but the saw must be used for beveled cuts. Some shops use the saw for both square and beveled cuts. This is a much slower and more expensive process than shearing plates and angles, and for that reason it is best to order them to the lengths wanted. They can be trimmed to exact length by the coping machine, which has a heavy square punch for removing portions of flanges, and also has shearing knives for cutting the web after the flanges are removed. Beveled cuts on beams and channels are expensive and troublesome. The smaller sizes of beams and channels can sometimes be beveled in the plane of their webs by first coping off one flange and then cutting the web and other flange on the angle shear. This, of course, means carry-

ing the stock from one machine to the other and handling it twice. Channels that are too large or too sharply beveled for the limits of the angle shear, and beams of all sizes, can have one flange removed by the coper and then have the web sheared to a bevel. This leaves them as at *A* in Fig. 79, with one flange cut square. This blunt corner will rarely be objectionable, and the draftsman should draw them that way when possible, instead of

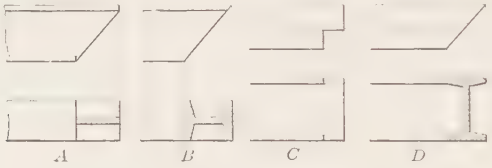


FIG. 79.—Different ways of beveling ends of I-beams and channels.

like sketch *B*. Beams or channels beveled transversely to their webs are especially objectionable to the shop. Instead of cutting the flanges on a bevel, they can be coped as shown by the sketch *C* in Fig. 79. When the engineer insists on a sloping cut, as at *D*, Fig. 79, the shop may be driven to heating the end of the beam in a furnace in order to make the cut by hand at a reasonable cost. In most cases this is certainly more objectionable than the sacri-

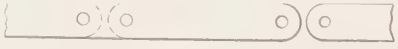


FIG. 80.—Method of cutting and punching lattice bars.

fice in appearance where the beam is coped like the illustration. When coping in this way to obtain clearance, it is not necessary to cut off the flange absolutely flush with the web. It is well to say "Cut flush" when this is necessary, and have it understood that an ordinary cut will do in other cases.

Lattice Bars.—Lattice bars are made by a punch that forms the rounded end and punches the hole on one end of each of two bars at one stroke of the machine, as shown by Fig. 80. A gage is set to give the length and no marking of the iron is necessary. The draftsman should therefore have as few kinds as possible and should avoid any special shape of ends.

Flange Plates.—Girder flange plates are frequently drawn with the corners cut at 45°. Where double gage lines are used in the angles and the rivet pitch is small, it is necessary to put

the end rivets of the flange plates on the inner gage lines and cut the corners to clear the rivets on the outer gage lines. But where the pitch is sufficient a better plan is to have the end rivets on the outer gage lines and cut the plates square. This not only saves a large amount of shop labor, but makes it much easier for the track man to frame his ties in the case of a deck railroad girder. As far as looks are concerned, the remarks on shearing angles square instead of beveled are applicable.

Straightening.—Long plates have to be straightened edgewise after delivery at the shops. If used for webs of riveted members they should be narrower than the distance back to back of flange angles by 1/4 to 1/2 in. to allow for slight crookedness. Much work on the straightening press will then be saved, as well as probable chipping on the member before the flange plates and gussets can be put on. There will also be less danger of damaging the steel by trying to get out small kinks. Web plates of girders having full length top plates to protect them from water should be ordered *one-half inch* narrow, or they will have to be sheared or chipped in places.

Punching.—When more than one size of holes is called for on a piece, that piece must be handled once for each size, and each time the cost of punching will be practically as great as would be the cost of punching all the holes the same size. This is very nearly true, even if only one hole of a different size is called for. Further, it costs as much to drive a five-eighths rivet as a seven-eighths rivet, and generally more of the smaller size will be required than of the larger. When driving the rivets a change in size means stopping the work to change dies, or else going over the work a second time. Thus, both punching and riveting considerations speak for uniformity in size of holes.

It will generally be cheapest to widen lattice bars that are so narrow as to require smaller rivets than the rest of the work. If some of the holes must be reamed for bolts, make the bolts 1/8 in. larger than the rivets. If more than one size of holes must be used in the job, make the change in the small pieces, as they can be handled easier from one punch to another. Holes for spiking-piece bolts and tie-rods, when smaller than those required by the rivets used in the connections, can be made the same size as the rest of the punching, if a washer is used under the nuts. It will often pay to increase the width of leg of some pieces that are designed with regard only to the stresses, in order to enable the reg-

ular size of rivets to be used. Holes larger than the usual maximum allowable diameter for a given size of beam or channel may be put into beam flanges at the ends of the pieces, where section is not required for strength. Slots in webs should be made of the same width as the diameter of the punch.

Uniform gages for all holes in the leg of an angle are very desirable. When but one gage line is used the templet consists of a pole that is laid beside the angles, and chalk lines are squared across from the marks on the pole. A gage is set on the punch, and the holes are punched without center punch marks being made. When flat templets are used and the holes marked with a center punch, the use of gages is still advantageous, aiding the men at the punch to do more rapid and accurate work. A hole not on the regular gage, however, must be specially marked and extra care taken to avoid punching it like the regular holes.

Punches are likely to break when of smaller diameter than the thickness of metal punched. Holes should not be put on a joint so as to make half holes in the abutting pieces, as the teat on the punch will force the piece to one side and make a bad hole, besides soon breaking the punch.

Assembling.—In some shops the men that mark the material for cutting and punching are supplied with the drawings. The templets in this case do not give size of holes, countersinking, allowance for facing, etc.; and the markers have to spend considerable time studying the drawings, and they are high-priced men. Other shops do this work in the templet shop and put all information needed by the marker on the templet. Then the drawings are not used in the shop until the punched material is assembled for the riveters.

The cost of assembling, or "fitting up," varies greatly with the character or diversity of the work and may be as great as the cost of punching. The men engaged in this work want clear drawings, with the various plans and elevations shown in proper relation to each other. End views rather than sections should be given, for a man can take his drawing around to the end of a piece and compare it with the work, but he cannot see a section without the use of a good deal of imagination. Top, bottom and end views should always be placed as shown in Fig. 81, and not with the top view below and the bottom view above. No shop man wants the right-hand end view of a piece at the left of its elevation, and no one else will that has ever tried to use such a drawing

in the shop. The one exception to this rule is that the bottom flange of a girder is generally shown by a sectional plan, and this is done more as a concession to the draftsman than by the wish of the shop.

Columns are assembled and riveted in a horizontal position and should be drawn in that way, with the bottom to the left. If drawn vertically every man in the templet department or shop will have to turn the drawing around when he uses it.

Each drawing should have on it a list of all the complete members shown on the sheet. This list should give the number wanted, name, hand, and shipping mark in this way:

Two End Posts wanted, as shown, Marked *LOU2R.*

Two End Posts wanted, opposite hand, Marked *LOU2L.*

The foreman checks off against this list as he fits up the work. The list should have a descriptive name for each piece as well as a shipping mark, and this name should also appear on the shop bills and shipping invoices. All notes should be written in language that is not ambiguous through being too much abbreviated; notes are often put on in such poor English that they can be interpreted in more than one way.

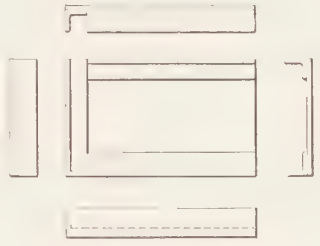


FIG. 81.—Proper arrangement of views in the representation of an object.

Open holes, flattened, and countersunk rivets should be shown in such a way that they can be marked for the riveters without having to be located by measurements on the iron. The required clear spaces between chords, and the extreme width of entering members should be given directly, and should not have to be obtained by adding up other figures. The distance between open holes on end connections should be given similarly. These distances have to be checked on the iron before riveting is begun. Wooden blocks are often bolted into the chords and clamps used on entering members to keep such pieces to the proper width while being riveted up. Note dimensions that must not vary over 1/16 in. as "exact," this being the practical limit of accuracy in structural work. Notes should be given positively, stating that a "Bracket goes on Columns 1, 2, 3, 4, and 6," and not "Omit bracket on Column 5 only." Small sketch views should be used when pieces shown on the

same drawing differ much in arrangement of attached parts. It is much cheaper to make them than to have the assembling gang losing time puzzling over complicated notes that are more or less hidden by paint and grease spots, due to handling the drawings and the steel work at the same time.

Center lines, gages, bevels, etc., do not appear on the punched material, and it is often difficult to tell from inspection which way a piece should go. The spacing of the shop rivets can often be made in a way to prevent reversing a gusset or riveting an end connection angle upside down. If the two legs of a connection angle are alike except for a slight difference in gage, the gage of the shop-driven leg should be changed to be either the same as the other leg, or else markedly different from it.

Riveting.—Shop rivets are generally calculated at higher values than field-driven rivets, it being assumed that they are driven by machine riveters capable of exerting heavy pressure. It is important, therefore, to avoid placing them in positions that cannot be reached by these machines. Rivets that cannot be reached by the machine riveter are left to be driven later by hand at a much greater cost.

Fig. 82 shows the factors determining gages and clearances for such machines when used with full-sized dies. When rivets must be driven on narrow gages the corner of the die is ground away to clear the fillet of the angle. In such cases the rivet head will very likely be forced to one side and will not be concentric with the shank, and the dies will soon lose their proper shape and will have to be repaired or renewed. For the same reason flange rivets should not be placed too close to stiffener angles or lateral gussets.

Countersunk or flattened rivets require the changing of dies, and if countersunk rivets have to be flush with the surface of the piece they must be chipped by hand. When they occur in shoe plates they are generally driven by hand and chipped flush while hot. As they are not required for carrying stresses in such places, their number can often be reduced to advantage.

Draftsmen have been known to place flange plate rivets directly under the outstanding leg of a stiffener, countersinking them in the angle. This practice is very objectionable, for a number of reasons. It involves countersinking the hole, thus removing more metal that was figured on by the designer, and the rivet must, of course, be chipped. The most serious objection,

however, has to do with the order in which the riveting is done. It is customary in plate girder work to first bolt the flange-angles, stiffeners, fillers and splice plates to the web and drive all the rivets whose axes are at right angles to the web. Then the flange-plates are put on and their rivets driven with a different machine. But if the flange-plate rivet comes under a stiffener leg, that stiffener will have to be left off until the flange rivets are

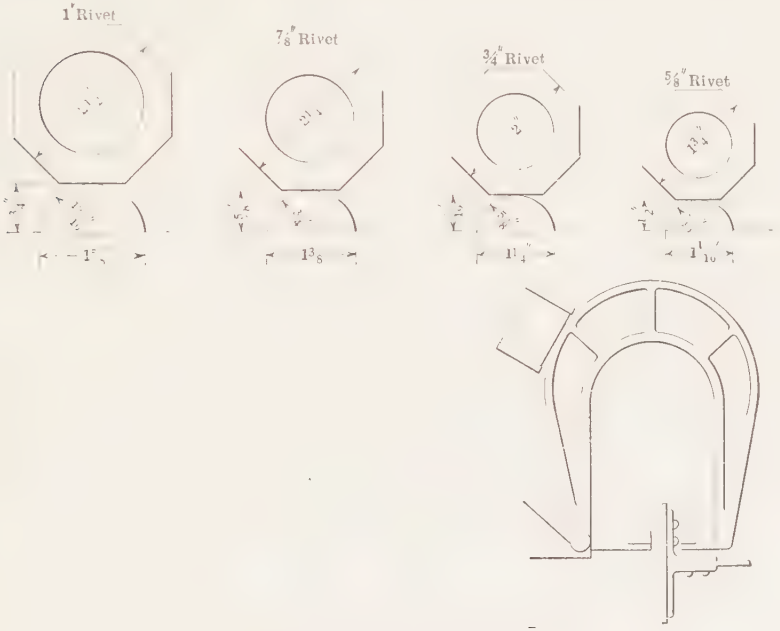


FIG. 82.—Sketches showing factors which determine minimum gage in angles, with rivets driven opposite.

driven. Then the girder will have to be taken back to the first machine for driving the stiffener rivets, or they will have to be driven by hand. For the same reason girder stiffeners should not be fitted down on to lateral gussets, but the gussets should be notched around the stiffener.

These examples will serve to illustrate the need of keeping in mind the order of operations in assembling and riveting. When it can be done, members are assembled complete before riveting is begun. Box chords will have their sides driven up first and then the cover-plates and lattice added. When pin plates are placed over the web legs of the angles they may interfere with driving

the cover plate rivets. Brackets on small columns frequently involve very troublesome riveting, and the draftsman must consider how the shop is going to accomplish what he calls for. Skew connections must often be laid out full size and a rivet of the required length be tried to see if it can be entered into the hole. Rivets in one leg of an angle may have to be countersunk or flattened in order to get rivets into the holes through the other leg. If there is barely room enough to get a rivet into a hole there is sufficient room so that it can be held on that end, but there must be room on the other end for driving it.

In building work it is frequently possible to so arrange connections that many of the pieces will have no shop riveting on them at all. For instance, a beam resting on a column bracket will have a lug on top connecting it to the column. If the lug is attached to the column the beam can go right from the punches to the shipping yard. Knees connecting purlins to trusses can be riveted to the trusses, and the same can be done with the bracing gussets.

Blacksmith Work. Blacksmith work is very expensive in comparison with the other features of structural work, and consequently should be avoided as much as possible. The cost will often depend very much on the amount of duplication. When there are enough bent plates of one kind a special cast-iron die block and follower can be made, and the actual bending done very rapidly, the cost of the die being small in comparison with the saving of labor. When only a few pieces of a kind are wanted the cost per piece may be ten times as great.

Curved end angles for plate girders should be made to fit standard formers and should always be made as separate pieces and spliced to the main flange angles a foot or so beyond the bend. Allow a foot or two of extra stock for each curved piece, as the blacksmith cannot tell just where the curve will come and must trim off the ends after bending. Do not have a bend at each end of a piece, as it is very difficult to get the two bends the right distance apart. The blacksmith will have to work over it so long that the quality of the material may be injured.

Do not call for a sharp bend on a plate if a radius of 1/2 in. or 3/4 in. can be allowed. Besides the additional labor involved, the metal on the outside of a sharp bend will be drawn away and the plate weakened.

Top chords for light roof-trusses of small pitch are sometimes

made with single lengths of angles bent at the apex. Anything of this kind is a nuisance to handle at the punches, and splicing is generally cheaper in spite of the extra material and rivets required. If such bends are made they should, if possible, be to a radius that will allow them to be bent cold in the press.

III. SHIPPING AND ERECTION

Shipping.—The maximum size of a single piece that is to be shipped by rail is fixed by the regulations of the roads over which it has to go. Anything not over 9 ft. wide and 40 ft. long can be loaded flat on a single car. Pieces up to 10 ft. wide can be loaded on edge on practically all roads, but if wider pieces are wanted the clearance limits of the roads over which they are to pass must be examined. When pieces are longer than a car, they must rest on bolsters about 6 in. high, which, of course, use up just that much of the available head room. These limits are based on cars of standard height, and should not be exceeded without investigation.

Roof trusses that it is desired to ship riveted up complete may be so near the available limit that some projecting gusset, purlin, lug, or splice, will cause them to exceed it. They must then be sent in pieces, or else the details must be modified to keep the truss within the proper size. The draftsman will have to balance the cost of riveting in the field against the cost of the same work in the shop with the perhaps more elaborate details. Field riveting requires more rivets and consequently larger gussets. The rivets will cost a good deal more to drive, say 5 cents each, for field, as compared to 2 cents each for shop. And if any errors are found when assembling in the field they are much more expensive to correct than they would have been if found in the shop. These considerations lead to the general rule to avoid all field riveting possible.

But there is another point to consider in many cases. Railroads charge for at least 36,000 lb. of freight per car, and if the actual load is less than this there is a corresponding increase in cost per pound for freight. A job composed of 25,000 or 30,000 lb. of columns or roof trusses that are 60 ft. long will then cost but half as much for freight if spliced in the field, and this may determine the proper course to pursue. Still smaller jobs, like foot bridges, amounting to three or four thousand pounds, that

would require a whole car if their trusses were riveted up in the shop, can be shipped knocked down through the freight office at regular pound rates. If the work has to be teamed from station to site, it will, of course, be easier to handle small pieces.

Long girders should be shipped so loaded that they will be pointed the right way when they reach the site. Sometimes, when the proper direction cannot be foretold, it is possible to leave off the shoe plates and thus make the girders themselves reversible. The erector must then rivet on the shoe plates before lowering the girders to place. Projecting gussets or angles on heavy members are liable to injury from having the weight of the member thrown on to them in handling. At least one side of heavy pieces should therefore be free from such projections.

Shipping lists are generally made by the draftsman, and should describe each piece, as well as give its shipping mark. The description should include the principal dimensions, and the items may read as follows:

Two Top Chords 24 in.×36 in.×58 ft., 6 in., Marked *U2U4*.

Four splice Plates, 16 in.×3/8 in.×2 ft., 0 in., Marked *B8*.

Etc., etc.

Great care must be taken to have all the separate pieces listed. It costs several days' wages to send by express a splice plate that the draftsman failed to list.

An extra allowance of rivets is required to replace those lost or spoiled in heating and driving; 20 per cent. is not too many to send additional, as those not used can be returned with the tools. The erector should be furnished an itemized list, showing where the rivets of different lengths are intended to go. If he finds he needs any different lengths, he will then know at once what to send for, and will not run out of sizes unexpectedly.

Erection.—Erection work is usually done under far less favorable circumstances than the other parts of the manufacture of a bridge. The men are exposed to the weather, and much of the work is done with temporary appliances that would not be considered adequate in the shop. Frequently an old bridge must be replaced by a new one while traffic is maintained, and the work is largely done at night and in a hurry. Sometimes the falsework is in danger from rising streams, and a few hours' delay may mean the loss of a span worth thousands of dollars. An error that, if found in the shop, could be easily and cheaply remedied, and perhaps would not even be reported to the drafting

room, will become a very costly and serious mistake when found in the field. A heavy truss member will be brought out between trains and after several hours' work be gotten nearly into place when it will be found that the draftsman has overlooked a rivet that should have been countersunk, and the member has to be taken out to get at it. The available time has expired and half a night's work has been wasted. Even if the rivet can be cut off without taking out the member, most of the gang may have to loaf while it is being done.

The most effective way, then, to reduce erection costs is to realize the seriousness of errors affecting that part of the work. The same reasons that make mistakes so costly make it worth while to plan carefully for the erectors' convenience in every way possible.

The general drawings of the work are designated as "erection plans," and should be complete enough for all ordinary purposes, the detail plans being kept in the tool-box for occasional reference. The erection plans should give the principal dimensions of the work and show the direction to the nearest important railroad station, street names, points of the compass, or some similar means of fixing the way the structure stands. An index of all the drawings should be given on one of the erection plans. The name of every separate piece that appears on the shipping list should be given in its proper place. Erection plans of truss bridges should give extreme height and width, these dimensions being needed to determine size of traveler. The spacing of the floor-beams fixes the location of the falsework bents. The clear headroom under portals and bracing, and the clear width between trusses, should appear; also the distance from masonry to center of end-pin, top of floor-beam, or some similar point that will govern the erectors' layout. As a general thing, it is not wise to economize by showing less than the whole length of a bridge span, even when it is symmetrical. For buildings these plans should give grades of all wall and shoe plates, as well as their location from the building lines. Where existing masonry has to be cut away, the plan should show size and location of holes, in order that they may be cut in advance by the mason.

A system of marking that is used with much success by some companies is as follows: The templet maker always writes his "shop mark" and orders on the left end of his templet, this being always the left end as shown by the detail drawings. The

marks are painted on the steel just as they appear on the templets. The name of each member on the erection plan is placed on the end that is to the left on the detail drawing. The result is that the erector has only to place the "marked end" of his piece to correspond with the mark on his drawing. The same system is followed out on all the pieces that make up a complete member, and is of great assistance in the shop. All templets are sent to the shop with the marked ends together, and as a result all punched material reaches the assembling gang with its marks one way, and the riveted members are sent out of the shop, stored in the yard, and finally shipped with a uniformity that is a help in many ways. In order to make this system still more effective, the draftsman should follow regular rules in drawing truss members, columns, etc., laying them out on his detail drawings with the left-hand end as the end nearest the abutment, or by some similar rule.

The arrangement of details for greatest economy and convenience in erection requires a knowledge of erection methods that is not common. More money can be made or lost in erection than in any other part of a job. The cost of the materials and of the drawing, templet and shop labor, the amounts to add for rent, fuel, office expenses and salaries, and the freight to the site, can all be determined in advance with a small percentage of error. The estimated cost of erection is generally a guess that is liable to vary 25 per cent. from the final result. This is due in part to unavoidable circumstances and in part to the fact that erection has been left so largely in the hands of the workmen unassisted by the engineering department. An engineer will see where details can be changed to help erection, and yet not injure the structure theoretically, but a foreman cannot go into that side of the question. The present tendency is to remedy this state of affairs by bringing the erection and engineering departments together for mutual advice and assistance. This, of course, is essential on great enterprises, and will undoubtedly be found profitable on all classes of work.

Erection of Building Work. Pedestals and base plates for columns, and wall plates for beams and girders, should be separate pieces, and the erection plans should give the grades of their top surfaces. They can then be set in advance to accurate grade and location. Columns should be bolted to the pedestals, as they will then need little guying while the beams are being put in place. All columns should be spliced at one level to avoid interfering

with the derricks that rest on the floors and are moved up a column length at a time. The splice should be high enough above the floor to admit of riveting after the floor is laid. The splice plates can be riveted to the lower section, but the rivets nearest the end of the section should be omitted to allow the splice plates to be spread a little in order to enter the upper section. If the vertical distance between the splice plate holes at the joint is made a sixteenth less than the distance between column holes, the sections will be drawn together when pinned up for riveting.

Beams are often connected to columns by resting on brackets, to which they are riveted through their lower flanges. Stiffness is secured by having knees on top of the beams connecting them to the columns. These knees are best shipped loose. They should be drawn with 1/8-in. clearance underneath, as beams nearly always overrun in height on one side, the top and bottom flanges not being square to the webs. When double beams connect to columns or to other beams, parts of their flanges may have to be removed at the ends in order to drive the connecting rivets. Long bolts going through both beams can be used, but are not very satisfactory. If knees are riveted to a beam to receive the web of another beam between them, the drawing should call for a space 1/16 in. greater than the web. A beam having this kind of a connection at both ends is likely to be hard to get in.

Erection plans should show which way the edges of channels turn, the line of the web being dotted in. They should also show the direction of the column webs and the sizes of the beams, as well as their names. All pieces that are identical should have the same mark. If every column has a separate number, the erector will presumably have to overhaul his pile to find the right one when there may be others on top that will do as well. As a matter of fact, he will generally try to find out from his details what ones are alike in order to save himself this trouble. And in that case he is, of course, likely to make mistakes. Tie-rods should be designated by their length, as they are too small for marking by paint.

Erection of Roof Work.—Roofs consisting of trusses connected by bracing or purlins can often be planned to avoid any riveting after the trusses are hoisted in place. The parts of the truss are assembled on the ground and riveted together and the truss hoisted as a whole, the bracing and purlin connections being

bolted. If rivets are required, a staging must be built at each joint, and the rivets may cost 25 cents apiece. One man can put in bolts without any staging, but riveting requires a platform to work on and four men in the gang. Common bolts are perfectly good in many places, and it is good engineering to take advantage of their cheapness in such cases. Turned bolts with a close fit in the holes may sometimes be necessary. If they are expected to supplement rivets in the same joints, they must fit the holes as tightly as the rivets. This will require a taper bolt, and the holes must be reamed for each one as it is put in. They will be very expensive and of doubtful value, and the draftsman should only call for them as a last resort. Places that are too confined to admit of riveting are generally too confined to admit of the proper reaming of holes and screwing up of bolts.

Erection of Plate Girder Bridges.—Plate girder bridges almost always have the girder shipped whole. In deck bridges the sway frames (see Plate III) should be 1/8 in. less in depth than the space they are to occupy. In through bridges having floor-beams and stringers, the floor may be put in place first and the girders moved in sideways, or the girders may be placed first. If the latter plan is followed, the floor-beam connection to the girder must be planned to allow the beams a movement along the girder that will separate them far enough to get in the stringers. The stringers have to be put in diagonally past the floor-beam flanges and then turned straight. The length of the stringer on its longest diagonal should be computed and the possibility of getting it in investigated fully.

One very common difficulty met with in erection is the drilling of the holes in the stone piers for the anchor bolts. This must be done after the steel is in place. The drills should be at least 1/4 in. larger than the bolts to make a hole that will admit the cement around the bolt. But draftsmen call for holes in the shoe plates that are just large enough to admit the bolt and place them under gussets and end sway frames, or through the lower angles of stringers, and seem to think the holes will be marked through on to the stone and the bridge removed while the holes are drilled. Consequently, erectors have to chip out the steel to get their drills in on a slant that will bring the top end where they can strike it. The result is a crooked bolt, or frequently no bolt at all. Anchor-bolt holes in shoe plates should be 1/2 in. larger than the bolts and in positions admitting of holding the drills

vertical, with room to swing a sledge for striking them. Use a washer under the nut to cover the hole.

Another common oversight is to call for field rivets in the bracing over abutments and piers where the stone is so near that the rivets cannot be entered. In fact, the bracing itself is not infrequently found to interfere with the masonry.

Erection of Highway Bridges. The erection plan for a highway bridge should fully show the woodwork for the floor and fence. If the bridge is on a skew, the ends of the planks at the abutments must be supported in some way. Similarly, wooden fences and wheel guards have to end somewhere, although draftsmen have a way of leaving such matters undetermined. Roadway plank should be planed on one side to obtain an even thickness, and will then measure less than the nominal thickness. Planks that must break joint should be all of one width. Wooden stringers must be sized to less than their nominal depth at the ends in order to bring all their tops to the proper level, and heights of shelf angles must be fixed accordingly. Wooden stringers will shrink, and where they are nearly flush on top with steel floor beams some provision must be made for carrying the planking over the beam without resting on it. Spiking pieces on top of beams are best attached by bolts that go entirely through the wood, for lag screws are much harder to use. All lumber to be painted should be ordered planed, and plans should state the number and sizes of nails required. The woodwork of a highway bridge is often the greater part of the work of erection, and a large item in the total cost of the job.

Erection of Truss Bridges.—Truss bridges are erected in two general ways. The floor beams and stringers may first be assembled on the falsework and the trusses bolted up to them as they are erected, the floor beams holding the trusses in place until the top bracing is put in; or the trusses may be put in first and the floor afterward. The draftsman should be informed in advance of the method that will be used.

Most of the remarks already made will apply to truss spans. In addition, the draftsman must provide suitable clearances for posts entering into chords and similar connections. Built members will vary a little from figured dimensions, being slightly out of square or measuring a little larger when several thicknesses are piled up together. Rivet heads will be higher than figured. Most offices have rules for clearances based on their own ex-

perience, but if none are at hand the draftsman may use the following:

Assume all eyebars and plates used singly as 1/16 in. thicker than figured. Where plates are riveted together, assume each one as 1/32 in. thicker than figured. Assume all countersunk chipped rivets as 1/8 in. high, and all flattened or full-head rivets as 1/16 in. higher than figured. Then add 1/8 in. on each side of a member for clearance as it is put into place.

The portals and overhead sway frames are put in last, and should be arranged to go in without spreading the trusses. It is often a good plan to ship them with the top angles separate. These top angles can then be used for temporary bracing; they leave headroom for the derrick cars, which are coming more and more into use for erection purposes.

Pins are driven into place with pilot nuts temporarily screwed on to the ends. Room must be left to get these nuts off when floor beams or bracing connections come opposite the ends of the pins. Room must be left around pin holes for the nut to turn. When the holes come near the edge of an angle, this often requires a filler to be riveted to the web plate of the chord.

Ends of chords and end posts with half holes that bear on pins should be cut to clear each other by 3/8 in. They should not be faced to bear on each other when a pin is used, for it is then necessary to bolt them rigidly together before boring the pin hole. This makes trouble in the shop, especially when the two members make an angle with each other, as at a hip joint. Pin holes are bored with a horizontal boring bar, the members resting on a table or skids.

Illustrative Example.—The drawing of the plate girder bridge shown on Plate I illustrates many of the points mentioned in this chapter. The lateral bracing angles are cut square. Clearance is provided at the top ends of the floor-beam connection angles to allow the floor-beams to be moved past the rivet heads of the girder flanges while the stringers are being put in. The end and interior stringers are so drawn that one templet can be made for the end stringer and then be used for the interior stringer by adding a few holes at the left end, each set of holes being properly marked. The stringer cast pedestals have their anchor-bolt holes placed outside the width of the stringer flanges so that the holes can be drilled in the masonry after the stringers are in place. The curved ends of the main girders are of large radius and of separate pieces spliced to the main flange members. Interior stiffeners and fillers are punched on center lines as the fillers will then not have to be straightened edgewise after punching. In explanation of this it should be understood that the operation of punching distorts the

material around a rivet hole. When the piece is narrow, as fillers often are, if the hole is not central the material between the hole and the edge of the piece will be stretched more on the thinner side and the piece will consequently be curved a considerable amount in its length. Rivet spacing is tied in at all points and is alike on both legs of both flanges as far as possible. All rivets have sufficient clearances for driving by machine riveters in the shop. The lateral bracing is shown with all holes fully dimensioned. This is required by some shops but is not necessary when the floor is used for laying out such members in their relative positions. It is doubtful also if sufficient metal is saved by coring out the girder pedestal castings to pay for the extra pattern work required.

Conclusion.—The draftsman who plans his work with reference to the needs of the shop and erection department, while carrying out the intentions of the designing engineer, is entitled to part of the credit for the completed work and may rightfully feel that he is an engineer and not merely a mechanic. This chapter does not exhaust the subject by any means, but may serve to point out the way to the man that wishes to make the most of his occupation.

GENERAL SPECIFICATIONS FOR STEEL RAILROAD BRIDGES[1]

PART FIRST—DESIGN

I. General

Drawings.

(1) Detail or general drawings of each structure will be furnished by the Railroad Company. When detail drawings are furnished the Contractor shall compare and verify all dimensions shown before proceeding with any part of the work. If any mistakes, discrepancies or omissions are discovered the Engineer shall be immediately notified and his correction obtained. Where the Railroad Company furnishes general drawings only, the Contractor shall prepare all stress sheets and detail drawings and submit them to the Engineer for his approval.

Shop Drawings.

(2) The Contractor shall prepare all shop drawings and erection diagrams and they shall be submitted to the Engineer and be approved by him before they are used. In general the Contractor shall submit two prints of each drawing for approval.

Drawings to be furnished by Contractor.

(3) The Contractor shall furnish the Engineer four prints of each approved drawing. On completion of the work the tracings of all drawings prepared by the Contractor shall become the property of and be delivered to the Railroad Company.

Responsibility of Contractor.

(4) The Contractor will be held responsible for all internal dimensions and for the proper assembling of all parts.

Kind of Materials.

(5) The material in the superstructure shall be structural steel, except rivets, and as may be otherwise specified.

[1] These specifications are those of the New York, New Haven and Hartford Railroad Company dated 1912, and are here reprinted by permission of Mr. W. H. Moore, engineer of bridges of that company.

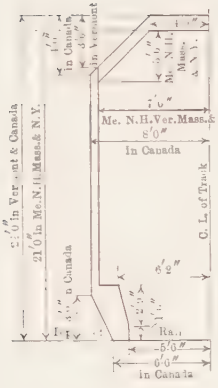
Clearance.

(6) On a straight line, clearances shall not be less than shown on the diagram. The additional clearance required on curves will be as follows:

$1.00 \times D$ = inches on each side.

$1.75 \times D$ = inches between tracks.

Where D = degree of curve.



For elevation the clearance at top of car on inside of curve must be increased three (3) inches for each inch of track elevation. The standard distance, center to center, of tracks on straight line will be thirteen (13) feet.

Spacing Trusses.

(7) The width, center to center, of girders and trusses shall in no case be less than one-twentieth of the effective span, nor less than is necessary to prevent overturning under the assumed lateral loading.

Skew Bridges.

(8) Ends of deck plate girders and track stringers of skew bridges at abutments shall be square to the track, unless a ballasted floor is used.

Timber Floors.

(9) Wooden tie floors shall be secured to the stringers and shall be proportioned to carry the maximum wheel load, with 100 per cent. impact distributed over three ties, with fiber strain not to exceed 2,000 lb. per sq. in. Ties shall not be less than 10 ft. in length. They shall be spaced with not more than 6-in. openings; and shall be secured against bunching.

II. Loads

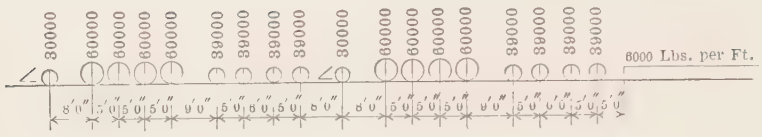
Dead Load.

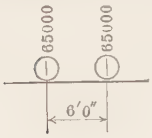
(10) The dead load shall consist of the estimated weight of the entire suspended structure. Timber shall be assumed to weigh 4 1/2 lb. per ft. B. M.; ballast 100 lb. per cu. ft.; re-enforced concrete 150 lb. per cu. ft.; and rails and fastenings 150 lb. per linear ft. of track.

Live Load.

(11) The live load for each track shall consist of two typical engines followed by a uniform load, according to Cooper's series, or a system of loading giving practically equivalent stresses. The following diagrams to be used:

Train Load.



and  the diagram that gives the larger stresses to be used.

Impact.

(12) The dynamic increment of the live load shall be added to the maximum computed live load stresses and shall be determined by the formula

$$I = S \frac{300}{L+300},$$

where

I = impact or dynamic increment to be added to live-load stresses.

S = computed maximum live-load stresses.

L = loaded length of track in feet producing the maximum stress in the member. For bridges carrying more than one track the aggregate length of all tracks producing the stress shall be used.

Impact shall not be added to stresses produced by longitudinal, centrifugal and lateral or wind forces.

Lateral Forces.

(13) All spans shall be designed for a lateral force on the loaded chord of 200 lb. per linear foot plus 10 per cent. of the specified train load on one track, and 200 lb. per linear foot on the unloaded chord; these forces being considered as moving.

Wind Force.

(14) Viaduct towers shall be designed for a force of 50 lb. per sq. ft. on one and one-half times the vertical projection of the structure unloaded; or 30 lb. per sq. ft. on the same surface plus 400 lb. per linear ft. of structure applied 7 ft. above the rail for assumed wind force on train when the structure is either fully loaded or loaded on either track with empty cars assumed to weigh 1,200 lb. per linear ft., whichever gives the larger stress.

Longitudinal Force.

(15) Viaduct towers and similar structures shall be designed for a longitudinal force of 20 per cent. of the live load applied at the top of the rail.

Centrifugal Force.

(16) Structures located on curves shall be designed for the centrifugal force of the live load applied at the top of the high rail. The centrifugal force shall be considered as live load and be derived from the speed in miles per hour given by the expression 60—2.5 D, where "D" is the degree of curve.

III. Unit Stresses and Proportion of Parts

Unit Stresses.

(17) All parts of structures shall be so proportioned that the sum of the maximum stresses produced by the foregoing loads shall not exceed the following amounts in pounds per sq, in., except as modified in paragraphs 23 to 26:

Tension.

(18) Axial tension on net section 16,000

Compression.

(19) Axial compression on gross section of columns ..... $16{,}000 - 70\frac{l}{r}$,
with a maximum of 13,500 lb.; where "l" is the length of member in inches, and "r" is the least radius of gyration in inches.

Direct compression on steel castings 16,000

Bending.

(20) Bending: on extreme fibers of rolled shapes, built sections, girders and steel castings; net section 16,000
on extreme fibers of pins 24,000

Shearing.

(21) Shearing: shop driven rivets and pins 12,000
field driven rivets and turned bolts 10,000
plate girder webs; gross section 10,000

Bearing.

(22) Bearing: shop driven rivets and pins 24,000
field driven rivets and turned bolts 20,000
expansion rollers; per linear inch ..... $600d$
where "d" is the diameter of the roller in inches.
granite masonry 600
Portland cement concrete 500
sandstone and limestone 400

Limiting Length of Members.

(22-*a*) The lengths of main compression members shall not exceed 100 times their least radius of gyration, and those for wind and sway bracing 120 times their least radius of gyration.

(22-*b*) The lengths of riveted tension members in horizontal or inclined positions shall not exceed 200 times their radius of gyration about the horizontal axis. The horizontal projection of the unsupported portion of the member is to be considered as the effective length.

Alternate Stresses.

(23) Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 70 per cent. of the smaller. The connections shall in all cases be proportioned for the sum of the stresses.

(24) Wherever the live and dead load stresses are of opposite character only two-thirds of the dead load stress shall be considered as effective in counteracting the live load stress.

Combined Stresses.

(25) Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the allowed axial stress.

Lateral And Other Stresses Combined.

(26) For stresses produced by longitudinal and lateral or wind forces combined with those from live and dead loads and centrifugal force, the unit stresses may be increased 25 per cent. over those given above; but the section shall not be less than that required for live and dead loads and centrifugal force.

Net Section at Rivets.

(27) In proportioning tension members the diameter of the rivet holes shall be taken 1/8 in. larger than the nominal diameter of the rivet.

Rivets.

(28) In proportioning rivets the nominal diameter of the rivet shall be used.

Net Section at Pins.

(29) Pin-connected riveted tension members shall have a net section through the pin-hole at least 25 per cent. in excess of the net section of the body of the member, and the net section back of the pin hole, parallel with

the axis of the member, shall be not less than the net section of the body of the member.

Plate Girders.

(30) Plate girders shall be proportioned either by the moment of inertia of their net section or by assuming that the flanges are concentrated at their centers of gravity; in which case one-eighth of the gross section of the web, if properly spliced, may be used as flange section. The thickness of web plates shall not be less than 1-160 of the unsupported distance between flange angles. (See 40, 81.)

Compression Flange.

(31) The gross section of the compression flanges of plate girders shall be not less than the gross section of the tension flanges; nor shall the stress per sq. in. in the compression flange of any beam or girder exceed $16{,}000\text{-}200\frac{l}{b}$, when flange consists of angles only, or if cover consists of flat plates; or $16{,}000\text{-}150\ \frac{l}{b}$, if cover consists of a channel section; where "l" = unsupported distance and "b" = width of flange.

Flange Rivets.

(32) The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange. The wheel loads, where the ties rest on the flanges, shall be assumed to be distributed over three ties.

Depth Ratios.

(33) Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams, used as girders, shall preferably have a depth of not less than one-twelfth of the span. If shallower trusses, girders or beams are used, the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratios had not been exceeded.

Provision for Waste by Corrosion.

(34) When bridges are over railroad tracks or subject to the action of salt water, the thickness of the following parts shall be increased 1/16 inch over that called for by the preceding rules to provide for waste by corrosion. Longitudinal through girders: web, two angles and one cover plate of bottom flange. Floor beams, stringers and girders wholly below the floor: web plates, and two angles and one cover plate of each flange. Trusses: Bottom chord angles and two web plates.

IV. Details of Design

General Requirements

Open Sections.

(35) Structures shall be so designed that all parts will be accessible for inspection, cleaning and painting.

Pockets.

(36) Pockets or depressions which would hold water shall have drain holes, or be filled with waterproof material.

Symmetrical Sections.

(37) Main members shall be so designed that each neutral axis will be as nearly as practicable in the center of section, and the neutral axes of intersecting main members of trusses shall meet at a common point.

Counters.

(38) Rigid counters are preferred; and where subject to reversal of stress shall preferably have riveted connections to the chords. Adjustable counters shall have open turnbuckles.

Strength of Connections.

(39) The strength of connections shall be sufficient to develop the full strength of the member even though the computed stress is less, the kind of stress to which the member is subjected being considered.

Minimum Thickness.

(40) The minimum thickness of metal shall be 3/8 in. except for fillers.

Pitch of Rivets.

(41) The minimum distance between centers of rivet holes shall be three and one-half diameters of the rivet. The maximum pitch in the line of stress for members composed of plates and shapes shall be 6 in. for 7/8-in. rivets and 5 in. for 3/4-in. rivets. For angles with two gage lines and rivets staggered the maximum shall be twice the above in each line. Where two or more plates are used in contact, rivets not more than 12 in. apart in either direction shall be used to hold the plates well together. In tension members composed of two angles in contact, a pitch of 12 in. will be allowed for riveting the angles together.

Edge Distance.

(42) The minimum distance from the center of any rivet hole to a sheared edge shall be 1-3/4 in. for 7/8-in. rivets and 1-1/2 in. for 3/4-in. rivets, and to a rolled edge 1-1/2 and 1-1/4 in., respectively. The maximum distance from any edge shall be eight times the thickness of the plate, but shall not exceed 6 in.

Maximum Diameter.

(43) The diameter of the rivets in any angle carrying calculated stress shall not exceed one-quarter the width of the leg in which they are driven. In minor parts 7/8-in. rivets may be used in 3-in. angles, and 3/4-in. rivets in 2-1/2-in. angles.

Long Rivets.

(44) Rivets carrying calculated stress and whose grip exceeds four diameters shall be increased in number at least one per cent. for each additional 1/16-in. of grip.

Pitch at Ends.

(45) The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets, for a length equal to one and one-half times the maximum width of member.

Compression Members.

(46) In compression members the metal shall be concentrated as much as possible in webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between its connections to the flanges. Cover plates shall have a thickness not less than one-fortieth of the distance between rivet lines.

Minimum Angles.

(47) Flanges of girders and built members without cover plates shall have a minimum thickness of one-twelfth of the width of the outstanding leg.

Tie-Plates.

(48) The open sides of compression members shall be provided with lattice and shall have tie-plates as near each end as practicable. Tie plates shall be provided at intermediate points where the lattice is interrupted. In main members the end tie-plates shall have a length not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones not less than one-half this distance. Their thickness shall be not less than one-fiftieth of the same distance.

Lattice.

(49) The latticing of compression members shall be proportioned to resist the shearing stresses corresponding to the allowance for flexure for uniform load provided in the column formula in paragraph 19 by the term $70\,\frac{l}{r}$. The minimum width of lattice bars shall be 2-1/2 in. for 7/8-in rivets, 2-1/4 in. for 3/4-in. rivets, and 2 in. if 5/8-in. rivets are used. The thickness shall be not less than one-fortieth of the distance between end rivets for single

lattice, and one-sixtieth for double lattice. Shapes of equivalent strength may be used.

(50) Three-fourths-inch rivets shall be used for latticing flanges from 2-1/2 to 3-1/2 in. wide; 7/8-in. rivets shall be used in flanges 3-1/2 in. and over, and lattice bars with at least two rivets shall be used for flanges over 5 in. wide.

(51) The inclination of lattice bars with the axis of the member shall be not less than 45 degrees, and when the distance between rivet lines in the flanges is more than 15 in., if single rivet bar is used, the lattice shall be double and riveted at the intersection.

Lattice. (Cont'd.)

(52) Lattice bars shall be so spaced that the portion of the flange included between their connections shall be as strong as the member as a whole.

Faced Joints.

(53) Abutting joints in compression members when faced for bearing shall be spliced on four sides sufficiently to hold the connecting members accurately in place. All other joints in riveted work, whether in tension or compression, shall be fully spliced.

Pin Plates.

(54) Pin-holes shall be reinforced by plates where necessary, and at least one plate shall be as wide as the flanges will allow and be on the same side as the angles. They shall contain sufficient rivets to distribute their portion of the pin pressure to the full cross-section of the member.

Forked Ends.

(55) Forked ends on compression members will be permitted only where unavoidable; where used, a sufficient number of pin plates shall be provided to make the jaws of twice the sectional area of the member. At least one of these plates shall extend to the far edge of the farthest tie-plate and the balance to the far edge of the nearest tie-plate, but not less than 6 in. beyond the near edge of the farthest plate.

Pins.

(56) Pins shall be long enough to insure a full bearing of all the parts connected upon the turned body of the pin. They shall be secured by chambered nuts or be provided with washers if solid nuts are used. The screw ends shall be long enough to admit of burring the threads.

(57) Members packed on pins shall be held against lateral movement.

Bolts.

(58) Where members are connected by bolts, the turned body of these bolts shall be long enough to extend through the metal. A washer at least

1/4-in. thick shall be used under the nut. Bolts shall not be used in place of rivets except by special permission. Heads and nuts shall be hexagonal.

Indirect Splices.

(59) Where splice plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number theoretically required to the extent of one-third of the number for each intervening plate.

Fillers.

(60) Rivets carrying stress and passing through fillers shall be increased 50 per cent. in number; and the excess rivets, when possible, shall be outside of the connected member.

Expansion.

(61) Provision for expansion to the extent of 1/8-in. for each 10 ft. shall be made for all bridge structures. Efficient means shall be provided to prevent excessive motion at any one point.

Expansion Bearings.

(62) Spans of 80 ft. and over resting on masonry shall have turned rollers or rockers at one end; and those of less length shall be arranged to slide on smooth surfaces. These expansion bearings shall be designed to permit motion in one direction only.

Fixed Bearings.

(63) Fixed bearings shall be firmly anchored to the masonry.

Rollers.

(64) Expansion rollers shall be not less than 6 in. in diameter. They shall be coupled together with substantial side bars, which shall be so arranged that the rollers can be readily cleaned. Segmental rollers shall be geared to upper and lower plates.

Bolsters.

(65) Bolsters or shoes shall be so constructed that the load will be distributed over the entire bearing. Spans of 80 ft. or over shall have hinged bolsters at each end.

Wall Plates.

(66) Wall plates may be cast or built up, and shall be so designed as to distribute the load uniformly over the entire bearing. They shall be secured against displacement.

Anchorage.

(67) Anchor bolts for viaduct towers and similar structures shall be long enough to engage a mass of masonry the weight of which is at least one and one-half times the uplift.

Inclined Bearings.

(68) Bridges on an inclined grade without pin shoes shall have the sole plates beveled so that the masonry and expansion surfaces may be level.

Floor Systems

Floor Beams.

(69) Floor beams shall preferably be square to the trusses or girders. They shall be riveted directly to the girders or trusses or may be placed on top of deck bridges.

Stringers.

(70) Stringers shall preferably be riveted to the webs of all intermediate floor beams by means of connection angles not less than 9/16 in. thick. Shelf angles or other supports provided to support the stringer during erection shall not be considered as carrying any of the reaction.

Stringer Frames.

(71) Where end floor beams cannot be used, stringers resting on masonry shall have cross frames near their ends. These frames shall be riveted to girders or truss shoes where practicable.

Bracing

Rigid Bracing.

(72) Lateral, longitudinal and transverse bracing in all structures shall be composed of rigid members.

Portals.

(73) Through truss spans shall have riveted portal braces rigidly connected to the end posts and top chords. They shall be as deep as the clearance will allow.

Transverse Bracing.

(74) Intermediate transverse frames shall be used at each panel of through spans having vertical truss members where the clearance will permit.

End Bracing.

(75) Deck spans shall have transverse bracing at each end proportioned to carry the lateral load to the support.

Laterals.

(76) The minimum size angle to be used in lateral bracing shall be 3-1/2 by 3 by 3/8 in. Not less than four rivets through the end of the angle shall be used at the connection.

(77) Lateral bracing shall be far enough below the flange to clear the ties.

Tower Struts.

(78) The struts at the foot of viaduct towers shall be strong enough to slide the movable shoes when the track is unloaded.

Plate Girders

Camber.

(79) If desired, plate girder spans over 50 ft. in length shall be built with camber at a rate of 1/16 in. per 15 ft. of length.

Cover Plates.

(80) Where flange plates are used, one cover-plate of each flange shall extend the whole length of the girder.

Web Stiffeners.

(81) There shall be web stiffeners, generally in pairs, over bearings, at points of concentrated loading, and at other points where the thickness of the web is less than 1–60 of the unsupported distance between flange angles. The distance between stiffeners shall not exceed that given by the following formula, with a maximum limit of six feet (and not greater than the clear depth of the web):

$$d = \frac{t}{40}(12{,}000 - s)$$

Where d = clear distance, between stiffeners or flange angles.

t = thickness of web.

s = shear per sq. in.

The stiffeners at ends and at points of concentrated loads shall be proportioned by the formula of paragraph 19, the effective length being assumed as one-half the depth of girders. End stiffeners and those under concentrated loads shall be on fillers and have their outstanding legs as wide as the flange angles will allow and shall fit tightly against them. Intermediate stiffeners may be offset or on fillers and their outstanding legs shall be not less than one-thirtieth of the depth of girder plus 2 in.

Stays for Top Flanges.

(82) Through plate girders shall have their top flanges stayed at each end of every floor beam, or in case of solid floors, at distances not exceeding 12 ft., by knee braces or gusset plates.

Trusses

Camber.

(83) Truss spans shall be given a camber by so proportioning the length of the members that the stringers will be straight when the bridge is fully loaded.

Rigid Members.

(84) Hip verticals and similar members, and the two end panels of the bottom chords of single track pin-connected trusses shall be rigid.

Eye-Bars.

(85) The eye-bars composing a member shall be so arranged that adjacent bars shall not have their surfaces in contact; they shall be as nearly parallel to the axis of the truss as possible, the maximum inclination of any bar being limited to one inch in 16 ft.

Pony Trusses.

(86) Pony trusses shall be riveted structures, with double webbed chords and shall have all web members latticed or otherwise effectively stiffened.

PART SECOND—MATERIAL AND WORKMANSHIP

V. Materials

Steel.

(87) Steel shall be made by the open-hearth process.

Properties.

(88) The chemical and physical properties shall conform to the following limits:

Elements considered	Structural steel	Rivet steel	Steel castings
Phosphorus, max. Basic	0.04 per cent.	0.04 per cent.	0.05 per cent.
Phosphorus, max. Acid	0.06 per cent.	0.04 per cent.	0.08 per cent.
Sulphur, maximum	0.05 per cent.	0.04 per cent.	0.05 per cent.
Manganese, maximum	0.55 per cent.	0.55 per cent.	0.55 per cent.
Ultimate tensile strength. Pounds, per square inch	Desired 60,000	Desired 50,000	Not less than 65,000
Yield point, minimum	55% Ult.	55% Ult.	50% Ult.
Elong., min. % in. 8″, Fig. A	1,500,000[1] / Ult. tensile strength	1,500,000 / Ult. tensile strength	15 per cent.
Elong., min, % in. 2″, Fig. B	22		
Character of fracture	Silky	Silky	Silky or fine granular
Cold bends without fracture	180° flat[2]	180° flat[3]	90°, $d = 3t$

[1] See paragraph 97. [2] See paragraphs 98, 99 and 100. [3] See paragraph 101.

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

(88-*a*) In order that the ultimate strength of full-sized annealed eye-bars may meet the requirements of paragraph 178, the ultimate strength in test

specimens may be determined by the manufacturers; all other tests than those for ultimate strength shall conform to the above requirements.

Allowable Variations.

(89) If the ultimate strength varies more than 4,000 lb. from that desired, a retest shall be made on the same gage, which, to be acceptable, shall be within 5,000 lbs. of the desired ultimate.

Chemical Analyses.

(90) Chemical determinations of the percentage of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analysis shall be made from finished material, if called for by the purchaser, in which case an excess of 25 per cent. above the required limits will be permitted.

Specimens.

(91) Specimens for tensile and bending tests for plates, shapes and bars shall be made by cutting coupons from the finished product, which shall

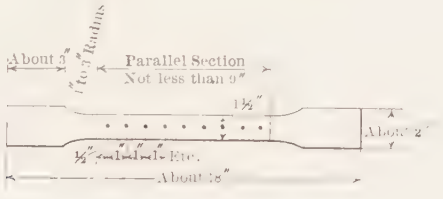


FIG. A.

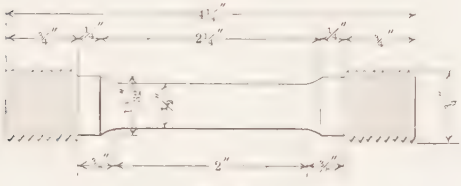


FIG. B.

have both faces rolled and both edges milled to the form shown by Fig. A; or with both edges parallel; or they may be turned to a diameter of 3/4 in. for a length of at least 9 in., with enlarged ends.

(92) Rivet rods shall be tested as rolled.

(93) Pin and roller specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be one inch from the surface of the bar. The specimen for tensile test shall be turned to the form shown by Fig. B. The specimen for bending test shall be one inch by 1/2 in. in section.

(94) For steel castings the number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons molded and cast on some portion of one or more castings from each melt or from the sink heads, if the heads are of sufficient size. The coupon or sink head, so used, shall be annealed with the casting before it is cut off. Test specimens to be of the form prescribed for pins and rollers.

Specimens of Rolled Steel.

(95) Rolled steel shall be tested in the condition in which it comes from the rolls.

Number of Tests.

(96) At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing 3/8 in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

Modifications in Elongation.

(97) A deduction of one per cent. will be allowed from the specified percentage for elongation, for each 1/8 in. in thickness above 3/4 in.

Bending Tests.

(98) Bending tests may be made by pressure or by blows. Plates, shapes and bars less than one inch thick shall bend as called for in paragraph 88.

Thick Material.

(99) Full-sized material for eye-bars and other steel one inch thick and over, tested as rolled, shall bend cold 180 degrees around a pin, the diameter of which is equal to one and one-quarter times the thickness of the bar, without fracture on the outside of bend.

Bending Angles.

(100) Angles 3/4 in. and less in thickness shall open flat, and angles 1/2 in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

Nicked Bends.

(101) Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky, uniform fracture.

Finish.

(102) Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish. Plates 36 in. in width and under shall have rolled edges.

Melt Numbers.

(103) Every finished piece of steel shall have the melt number and the

name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached metal tag.

Defective Material.

(104) Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

Variation in Weight.

(105) A variation in cross-section or weight of each piece of steel of more than 2-1/2 per cent. from that specified will be sufficient cause for rejection, except in case of sheared plates, which will be covered by the following permissible variations, which are to apply to single plates, when ordered to weight.

(106) Plates 12-1/2 lb. per sq. ft. or heavier.

(a) Up to 100 in. wide 2-1/2 per cent. above or below the prescribed weight.

(b) One hundred inches wide and over, 5 per cent. above or below.

(107) Plates under 12-1/2 lb. per sq. ft.

(a) Up to 75 in. wide, 2-1/2 per cent. above or below.

(b) Seventy-five inches and up to 100 in. wide, 5 per cent. above or 3 per cent. below.

(c) One hundred inches wide and over, 10 per cent. above or 3 per cent. below.

(108) Plates, when ordered to gage, will be accepted if they measure not more than 0.01 in. below the ordered thickness.

(109) An excess over the nominal weight, corresponding to the dimensions on the order, will be allowed for each plate, if not more than that shown in the following table, 1 cu. in. of rolled steel being assumed to weigh 0.2833 lb.:

Thickness ordered	Nominal weights	Width of plate			
		Up to 75 in.	75 and up to 100 in.	100 and up to 115 in.	Over 115 in.
Inch	Pounds	Per cent.	Per cent.	Per cent.	Per cent.
$\frac{1}{4}$	10.20	10	14	18	.....
$\frac{5}{16}$	12.75	8	12	16	.....
$\frac{3}{8}$	15.30	7	10	13	17
$\frac{7}{16}$	17.85	6	8	10	13
$\frac{1}{2}$	20.40	5	7	9	12
$\frac{9}{16}$	22.95	$4\frac{1}{2}$	$6\frac{1}{2}$	$8\frac{1}{2}$	11
$\frac{5}{8}$	25 50	4	6	8	10
Over $\frac{5}{8}$	.....	$3\frac{1}{2}$	5	$6\frac{1}{3}$	9

Cast-Iron.

(110) Except where chilled iron is specified, castings shall be made of tough gray iron, with sulphur not over 0.10 per cent. They shall be true to pattern, out of wind and free from flaws and excessive shrinkage. If tests are demanded, they shall be made on the "Arbitration Bar" of the American Society for Testing Materials, which is a round bar 1-1/4 in. diameter and 15 in. long. The transverse test shall be made on a supported length of 12 in. with load at middle. The minimum breaking load so applied shall be 2,900 lb., with a deflection of at least 1/10 in. before rupture.

Wrought-Iron.

(111) Wrought-iron shall be double-rolled, tough, fibrous and uniform in character and entirely free from steel scrap. It shall be thoroughly welded in rolling and be free from surface defects. When tested in specimens of the form of Fig. A, or in full-sized pieces of the same length, it shall show an ultimate strength of at least 50,000 lb. per sq. in., an elongation of at least 18 per cent. in 8 in., with fracture wholly fibrous. Specimens shall bend cold, with the fiber, through 135 degrees, without sign of fracture, around a pin the diameter of which is not over twice the thickness of the piece tested. When nicked and bent, the fracture shall show at least 90 per cent. fibrous.

Cast Steel.

(112) Steel for castings may be made by the open hearth or crucible process. All castings shall be annealed unless otherwise specified.

(113) Phosphorus..... 0.05 per cent. maximum.
Sulphur..... 0.05 per cent. maximum.

(114) Minimum physical qualities as determined on a standard test specimen of 1/2 in. diameter and 2 in. gaged length:

Tensile strength, in pounds per sq. in.....	70,000
Elongation: percentage in 2 in.....	18
Contraction of area: percentage.....	25

(115) A test to destruction may be substituted for the tensile test, in the case of small or unimportant castings, by selecting three castings from a lot. This test shall show the material to be ductile, free from injurious defects, and suitable for the purpose intended. A lot shall consist of all castings from the same melt or blow, annealed in the same furnace charge.

(116) Large castings shall be suspended and hammered all over. No cracks, flaws, defects, or weakness shall appear after such treatment.

(117) A specimen (1 in. by 1/2 in.) shall bend, cold, around a diameter of 1 in. through an angle of 90°, without fracture on the outside of the bent portion.

(118) The number of standard test specimens shall depend upon the char-

acter and importance of the castings. A test piece shall be cut cold from a coupon to be moulded and cast on some portion of one or more castings from each melt or blow, or from the sink-heads (in case heads of sufficient size are used). The coupon or sink-head must receive the same treatment as the casting or castings, before the specimen is cut out, and before the coupon or sink-head is removed from the casting.

(119) Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether or not the steel is within the limits in phosphorus and sulphur specified in Paragraph 113.

(120) Castings shall be true to pattern, free from blemishes, flaws or shrinkage cracks. Bearing surfaces shall be solid, and no porosity shall be allowed in positions where the resistance and value of the casting for the purpose intended will be seriously affected thereby.

Steel Forgings.

(121) Steel forgings may be made by the open-hearth or crucible process and all forgings shall be annealed.

Phosphorus	0.04 per cent. maximum.
Sulphur	0.04 per cent. maximum.
Tensile strength in pounds per sq. in.	55,000 to 65,000.
Elongation, 28 percent in 2 in.	

(122) A specimen (1 in. by 1/2 in.) shall bend, cold, 180° around a diameter of 1/2 in. without fracture.

(123) Forgings shall be free from cracks, flaws, seams or other injurious imperfections, shall conform to the dimensions called for, and shall be made and finished in a workmanlike manner.

Steel Discs.

(124) Steel discs, friction rollers, roller bearings and ball bearings for center bearings of drawbridges and turntables shall be of hammered open-hearth or crucible steel containing not less than 1.00 per cent. of carbon, and not over 0.04 per cent. of phosphorus nor over 0.04 per cent. of sulphur and not over 0.50 per cent. of manganese.

After being turned they must be case hardened and then ground to true curve.

Phosphor Bronze.

(125) Phosphor-bronze for bearings under high pressures shall have a minimum elastic limit in compression of 27,000 lb. per square inch. A 1-in. cube under a load of 100,000 lb. must not compress more than 1/16 of an in. A test piece shall be cut from a coupon to be moulded and cast on some portion of each casting. Test-pieces shall be 1 in. cubes, finished.

Manganese Bronze.

(125*a*) If manganese-bronze is used, it shall have a minimum elastic limit in compression of 28,000 lb. per sq. in., and the permanent set in a 1-in. cube under a load of 100,000 lb. must be not more than 1/10 of an inch.

Babbitt Metal.

(126) Where Babbitt metal is specified it shall have the following composition: Tin, two parts; zinc, one part; antimony, 5 per cent. of the weight of the tin and zinc.

VI. Inspection and Testing at the Mills

Mill Orders.

(127) The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled, nor work done, before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

Facilities for Inspection.

(128) The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of all material at the mill where it is manufactured. He shall furnish a suitable testing machine for testing the specimens as well as prepare the pieces for the machine, free of cost.

Access to Mills.

(129) When an inspector is furnished by the purchaser to inspect material at the mills, he shall have full access, at all times, to all parts of mills where material to be inspected by him is being manufactured.

VII. Workmanship

General.

(130) All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works. Material arriving from the mills shall be protected from the weather and shall have clean surfaces before being worked in the shops.

Straightening.

(131) Material shall be thoroughly straightened in the shop, by methods that will not injure it, before beng laid off or worked in any way.

Finish.

(132) Shearing and chipping shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

Size of Rivets.

(133) The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

Rivet Holes.

(134) When general reaming is not required the diameter of the punch shall be not more than 1/16 in. greater than the diameter of the rivet; nor the diameter of the die more than 1/8 in. greater than the diameter of the punch. Material more than 3/4 in. thick shall be drilled from the solid.

Punching.

(135) Punching shall be accurately done. Drifting to enlarge unfair holes will not be allowed. If the holes must be enlarged to admit the rivet, they shall be reamed. Poor matching of holes will be cause for rejection.

Reaming.

(136) Where sub-punching and reaming are required, the punch used shall have a diameter not less than 3/16 in. smaller than the nominal diameter of the rivet. Holes shall then be reamed to a diameter not more than 1/16 in. larger than the nominal diameter of the rivet. Reaming shall be done with twist drills and without the use of any lubricant. (See 151.)

(137) When general reaming is required, it shall be done after the pieces forming one built member are assembled and so firmly bolted together that the surfaces shall be in close contact. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be permitted.

Edge Planing.

(138) Sheared edges shall be planed at least 1/8 in. when such edges occur in web plates of girders, side plates of chords or posts, or in the cover plates of chords, posts or girders.

Burrs.

(139) The outside burrs on reamed holes shall be removed to the extent of making a 1/16-in. fillet.

Assembling.

(140) Riveted members shall have all parts well pinned up and firmly drawn together with bolts, before riveting is commenced. Contact surfaces to be painted. (See 168.)

Lattice Bars.

(141) Lattice bars shall have neatly rounded ends, unless otherwise called for.

Web Stiffeners.

(142) Stiffeners shall fit neatly between flanges of girders. Where tight

fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with flange angles.

Splice Plates and Fillers.

(143) Web splice plates and fillers under stiffeners shall be cut to fit within 1/8 in. of flange angles.

Web Plates.

(144) Web plates of girders, which have no cover plates, shall be flush with the backs of angles or project above the same not more than 1/8 in., unless otherwise called for. When web plates are spliced, not more than 1/4 in. clearance between ends of plates will be allowed.

Floor-beams and Stringers.

(145) The main sections of floor-beams and stringers shall be milled to exact length after riveting and the connection angles accurately set flush and true to the milled ends. If required by the Railroad Company, the milling shall be done after the connection angles are riveted in place, milling to extend over the entire face of the member. The removal of more than 3/32 in. from the thickness of the connection angles will be cause for rejection.

Riveting.

(146) Rivets shall be uniformly heated to a light cherry red heat in a gas or oil furnace so constructed that it can be adjusted to the proper temperature. They shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving. Rivets must have full hemispherical heads truly concentric with the shank. Rivets made in worn dies and exhibiting any lips, fins or fillets on head or shank will be rejected.

(147) Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjacent metal. If necessary, they shall be drilled out.

Turned Bolts.

(148) Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts shall make a driving fit with the threads entirely outside of the holes. A washer not less than 1/4 in. thick shall be used under nut.

Members to be Straight.

(149) The several pieces forming one built member shall be straight and

fit closely together, and finished members shall be free from twists, bends or open joints.

Finish of Joints.

(150) Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

Floorbeam and Stringer Connections.

(151) Holes for floor beam and stringer connections shall be sub-punched and reamed according to paragraph 136 to a steel templet not less than one and one-quarter inches thick.

Eye-Bars.

(152) Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to conform to the requirements of paragraph 178. The thickness of head and neck shall not vary more than 1/16 in. from that specified. (See 178.)

Boring Eye-Bars.

(153) Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin-holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that when placed together, pins 1/32-in. smaller in diameter than the pin-holes can be passed through the holes at both ends of the bars at the same time without forcing.

Pin Holes.

(154) Pin-holes shall be bored true to gages, smooth and straight, at right angles to the axis of the member and parallel to each other, unless otherwise called for. The boring shall be done after the member is riveted up.

(155) The distance center to center of pin-holes shall be correct within 1/32 in., and the diameter of the holes not more than 1/50 in. larger than that of the pin, for pins up to 5 in. diameter, and 1/32 in. for larger pins.

Pins and Rollers.

(156) Pins and rollers shall be accurately turned to gages and shall be straight and smooth and entirely free from flaws.

Screw Threads.

(157) Screw threads shall make tight fits in the nuts and shall be U. S.

standard, except above the diameter of 1-3/8 in., when they shall be made with six threads per inch.

Annealing.

(158) Steel, except in minor details, which has been partially heated shall be properly annealed.

Steel Castings.

(159) Al. steel castings shall be free from large or injurious blow holes and shall be annealed.

Welds.

(160) Welds in steel will not be allowed.

Bed Plates.

(161) Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The finishing cut of the planing tool shall be fine and shall correspond with the direction of expansion.

Pilot Nuts.

(162) Pilot and driving nuts shall be furnished for each size of pin, in such numbers as may be ordered.

Field Rivets.

(163) Field rivets shall be furnished to the amount of 15 per cent. plus ten rivets in excess of the nominal number required for each size.

Shipping Details.

(164) Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

Weight.

(165) The scale weight of every piece and box shall be marked on it in plain figures.

Finished Weight.

(166) Payment for pound price contracts shall be by scale weight. No allowance over 2 per cent. of the total weight of the structure as computed from the plans will be allowed for excess weight.

VIII. Shop Painting

Cleaning and Painting.

(167) The steel work before leaving the shop shall be thoroughly cleaned and given one good coating of pure linseed oil and powdered red lead, the mixture to be four pounds of red lead to one pint of oil. The paint shall be mixed fresh for the work and no more shall be mixed than is required for immediate use, as paint which has set or stood over night in pots must not be used. The paint must be well worked into all joints and open spaces.

Contact Surfaces.

(168) In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

Inaccessible Surfaces.

(169) Pieces and parts which are not accessible for painting after erection, including tops of stringers, eye-bar heads, ends of posts and chords, etc., shall have an additional coat of paint before leaving the shop.

Condition of Surfaces.

(170) Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

Machine Finished Surfaces.

(171) Machine-finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

IX. Inspection and Testing at the Shops

Facilities for Inspection.

(172) The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of workmanship at the shop where material is manufactured. He shall furnish a suitable testing machine for testing full-sized members, if required.

Starting Work.

(173) The purchaser shall be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship.

Access to Shop.

(174) When an inspector is furnished by the purchaser, he shall have full access, at all times, to all parts of the shop where material under his inspection is being manufactured.

Accepting Material.

(175) The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

Shipping Invoices.

(176) Complete copies of shipping invoices shall be furnished to the purchaser with each shipment. These shall show the scale weights of individual pieces.

X. Full-sized Tests

Eye-Bar Tests.

(177) Full-sized tests on eye-bars and similar members, to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected.

(178) In eye-bar tests, the minimum ultimate strength shall be 55,000 lbs. per sq. in. The elongation in 10 ft., including fracture, shall be not less than 15 per cent. Bars shall break in the body and the fracture shall be silky or fine granular, and the elastic limit as indicated by the drop of the mercury shall be recorded. Should a bar break in the head and develop the specified elongation, ultimate strength and character of fracture, it shall not be cause for rejection, provided not more than one-third of the total number of bars break in the head. (See 152.)

Section XI. Erection

General.

(179) All plans and detail drawings necessary for ascertaining the character and dimensions of the work will be at the disposal of the Contractor for erection. The method of erection will, in general, be left to the discretion of the Contractor, but no work shall be commenced until such method has been submitted to and approved by the Chief Engineer.

Work Included.

(180) The erection shall include the work of receiving and unloading all material, the transfer of the same from the place of storage to the bridge site (unless specified otherwise in contract), the drilling of the masonry and the setting of the anchor bolts, the building of the necessary falsework (See Paragraph 182), the erection of the new bridge complete in place, ready for the ties, or for the ballast in the case of solid floor bridges, the removal and reloading of the falsework and, if so directed, the careful taking down and loading on cars of the old bridge.

Handling and Storage.

(181) The Contractor shall unload all cars during the time allowed by the Division Engineer. The Railroad Company will deliver the material as soon as possible after it is turned over to it, but will not be held responsible for any delays caused by accidents, storms, floods, etc., while in transit over its lines. Cars must be released promptly upon their delivery, or the Contractor will be required to pay regular demurrage charges. Before erection, all bridge material shall be laid on skids above the ground so as to

be kept clean. Pieces such as girders must be laid on edge so as not to hold water. If any piece becomes soiled it shall be thoroughly cleaned by the Contractor. The steel shall be so stored and handled as to avoid injury to the material or interference with the Railroad Company's business. Any piece showing the effects of rough handling at any time during the progress of the work may be rejected.

Maintaining Traffic.

(182) Where traffic is to be maintained on the line of the bridge during erection, the Railroad Company will, in general, build the trestle or false work necessary to carry trains and will remove the same on completion of the work. Any changes required in this trestle or falsework to accommodate the erection of the bridge shall be made by the Contractor under the direction of the Engineer. The Contractor shall remove all timber, necessary for putting in the metal work, with as little damage as practicable and leave the same on the bank convenient for loading. In special cases, where the Contractor is required to build the temporary structure for maintaining traffic during construction, it shall be designed and built to the satisfaction of the Chief Engineer and removed by the Contractor on the completion of the work.

Manner of Erection.

(183) The Contractor shall conduct all his work so as not to interfere with the safe and uninterrupted passage of trains, the safe operation of Railroads and use of streets over, under or near the structure, nor with the rights of navigation in any river.

(184) The work of erection shall at all times be subject to the inspection and acceptance of the Railroad Company and shall be carried out in a first class, workmanlike manner and with foreman, force of men and plant satisfactory to the Chief Engineer.

(185) Any necessary side tracks or changes in existing tracks will be made by the Railroad Company, and shall be at the Contractor's expense if they are solely for his accommodation.

Laying Track.

(186) The Railroad Company will lay the ties, rails and guard rails, or, in the case of solid floor bridges, the ballast, ties and rails.

Anchor Bolts.

(187) Except when the anchor bolts are built up with the masonry, holes 1/2 in. larger than the diameter of the bolts shall be drilled by the Contractor for all anchor bolts after the metal is in place, and the anchor bolts shall be set in Portland cement grout.

Fitting Up.

(188) In fitting up the work, preparatory to riveting, the parts must be thoroughly drawn together with a very liberal number of bolts and drift pins drawing all the bolts up tightly with an even strain to avoid local injury to the material.

(189) Drift pins must not be used to distort the metal. Unfair holes must be reamed out with a tapered fluted reamer. If holes are much out of match they must be reamed for a larger rivet than called for on the drawings.

Field Rivets.

(190) Field rivets shall be tight and must have both heads well centered with the axis. The heads shall be of uniform size, full, without fins, concentric, closed tight to the metal and of as nearly as possible the same dimensions as those of the shop rivets.

(191) When the driving of tight field rivets is impossible, tight fitting turned bolts may be used when permitted by the Chief Engineer. Where such bolts are used they must be effectively locked by checking the threads, or a lock nut acceptable to the Chief Engineer may be used. (See 148.)

(192) No rivets shall be driven in the splices of compression chords or trestle posts until the abutting surfaces have been brought into a full and satisfactory contact throughout and submitted to the full dead load stress of the members. When the parts are required to carry traffic, important connections, such as attachments of stringers and floor beams, shall have at least 50 per cent. of the holes filled with bolts and 25 per cent. with drift pins; but tension splices shall be riveted up complete before the blocking is removed.

(193) Rivets must be heated uniformly throughout to a bright cherry red, no black heads or shanks will be allowed. Rivets must not be burned or overheated so as to spit when taken from fire. They must be driven immediately, the head being held up firmly to the work while the shank is being upset and closed down with a heavy hammer.

(194) All loose or defective rivets must be immediately cut out, care being taken not to injure the adjacent metal. If so ordered by the Inspector defective rivets must be drilled out. Recupping or calking of loose rivets will not be allowed.

Shop Errors.

(195) Any error in shop work which may be found during erection shall be promptly reported to the Chief Engineer, and shall be corrected at the Fabricator's expense.

Painting.

(196) The Contractor shall paint with one heavy coat all surfaces inac-

cessible after erection, including the tops of the stringers and other parts covered by the floor timbers; and the heads of all field rivets as soon as they are accepted by the Inspector. The paint for this purpose will be as specified in paragraph 167.

After erection the Railroad Company will paint the structure.

Old Structure.

(197) Unless otherwise specified the Contractor shall take down and load on cars the old structure, if any exist, in such a manner as to damage it as little as possible and preserve it intact for possible future erection, and before taking down shall mark all members in accordance with a diagram to be furnished him by the Chief Engineer. Should the old structure consist of several spans, the material in each span shall be kept separate.

(198) The Contractor will be held responsible for any damage done to material in taking down or handling the old structure and damaged members shall be repaired or, if found necessary, renewed at his expense.

Engine Service.

(199) No work train or engine service will be furnished to the Contractor free of charge unless the contract specifically provides for such free service. When derrick cars are used on main tracks, their movements shall be in charge of a train crew and the expense of this crew and any engine service shall be paid for by the Contractor, except where free service is provided for in the contract.

Removal of Falsework, etc.

(200) When the erection of the structure is completed the Contractor shall remove all his falsework and staging, clean up all débris and leave the site in as good condition as he found it.

Flagmen, Watchmen.

(201) The Contractor shall protect traffic and his work by flagmen, furnished by the Railroad Company at the expense of the Contractor, and shall provide competent watchmen to guard the work and material against injury.

Accidents.

(202) The Contractor shall assume all risks of accidents to men or material prior to the acceptance of the finished structure.

CONVENTIONAL SIGNS

A—Bridge Rivets

	Shop.	Field.
Two Full Heads.		
Countersunk and Chipped, far side.		
Countersunk and Chipped, near side.		
Conutersunk and Chipped, both sides.		

	Far Side	Near Side	Both Sides
Countersunk and not Chipped.			
Flattened to 1/4 in. high for 1/2-in. and 5/8-in. rivets.			
Flattened to 3/8 in. high for 3/4-in., 7/8-in. and 1-in. rivets.			

B—Stresses

+Tension.
−Compression.

EXPLANATION OF TABLES

Table No. 1 is a moment diagram for Cooper's E 60. The loads and distances between them are given as well as the sum of the moments of all loads from the left end up to any given load about any other given load. The following problem illustrates the use of the diagram. To find the moment of the loads 5 to 14 inclusive about 14, proceed as follows: The moment of all loads to the left of 14 is 13092. The moment of loads 1 to 4 inclusive about 14 is 7125. The moment desired is 13092 — 7125 = 5967. If the moment of these same loads is desired about a point say 2 ft. to the right of 14, add to the 5967 already found the product of the loads 5 to 14 by 2 ft. or 243 × 2 = 486. 5967 + 486 = 6453, which is the moment of all loads 5 to 14 inclusive about a point 2 ft. to the right of 14.

Table No. 2 gives gross areas of web plates for various depths and thicknesses. If the required area of the web be computed, enter the table with the known depth of web and follow horizontally to the right until the required area or the area next larger than that required is reached. The heading of this column will give the required thickness. The table can also be used to find the web equivalent 1/10*th* when it is desired to use that value. Entering the table with the given depth and thickness take 1/10 of the corresponding tabular value.

Tables Nos. 3 and 4 give web equivalents or the area of web which can be counted upon as flange area. Entering the proper table with the thickness and depth of web of the girder in question, the area of the web which can be counted upon as flange area may be found at once. Whether table 3 or 4 should be used depends upon whether one wishes $\frac{1}{8}$ or $\frac{1}{12}$ of the web to be counted as flange area.

Tables 5 and 6 give gross and net areas of pairs of angles in square inches. The weights of the angles are given in pounds per foot of length and are for two angles. The numbers in parentheses (2) or (4) at the heads of the columns indicate the total number of rivet holes taken out of both angles. In general (2) should be used where there are no cover plates and (4) where there are cover plates.

Table 7 gives the gross areas of cover plates for different widths and thicknesses. The widths vary from 8 ins. to 24 ins.; and the thicknesses from 5 16 in. to 5 ins. by sixteenths of an inch. If the area required and the width to be used are known, the total thickness of cover plates may be found at once from the table.

Table 8 gives the net areas of cover plates allowing for two $\frac{7}{8}$ in. rivets in a section.

Table 9 gives the net areas of cover plates of various widths allowing for two $\frac{3}{4}$ in. and two 1 in. rivets in a section. In both tables 8 and 9 the rivet holes are computed as $\frac{1}{8}$ in. larger than the diameter of the rivet.

Table 10 is an alignment chart by which it is possible, given any three of the four elements, flange area, moment, fibre stress and effective depth, to determine the fourth by laying two straight edges across the diagram. One straight edge must pass through the area and depth corresponding to the case under consideration and the other must pass through the proper fibre stress and moment. The two straight edges (or lines) must intersect on the line marked "area multiplied by depth." (See Table 10 and the problem at the end of the explanation of tables for a specific case.)

Table 11 gives the moment of inertia of webs of various depths and thicknesses. The axis used is the central axis crossing the web and the moment of inertia is computed from the formula $1/12th^3$ where t is the thickness and h is the height of the web. For the moment of inertia of the net area of the web use $\frac{3}{4}$ of the tabular value for the given web. This will give a result which is almost exactly correct for rivet holes spaced at the minimum permissible spacing (3 diameters) throughout the full depth of the web.

Tables 12 to 22 inclusive give the moments of inertia of various angles for various distances back to back. It is believed that the range of the tables will cover all cases met with ordinarily in practice. The quantities have been carefully calculated and checked. They are subject to an error not exceeding 5 units in the fifth significant figure. As the percentage of variation of weight allowed by the most severe specifications is much greater than this the tables are more accurate than practically required. The tables are calculated for gross area and for 4 angles, that is for 2 angles in each flange, so that the number taken from the table will give the moment of inertia about an axis at the half depth of the girder of all the angles in both flanges for girders having flanges of the type shown in Fig. 59 a, b, c or e. To obtain the moment of inertia of the net section of the angles, multiply the tabular value by the percentage which corresponds to the size and number of rivet holes in the section and which is given at the beginning of each table. It did not seem necessary to give a complete table for the 8 in. x 6 in. angle. To obtain the moment of inertia of the gross section of the 8 in. x 6 in. angle, enter Table 21 (diagram) with the depth of the girder back to back of angles, go up to the curve and across to the percentage sought. Multiply the moment of inertia of an 8 in. x 8 in. angle for the given thickness of angle and distance back to

back by the indicated percentage, and the result will be the moment of inertia of four 8 in. x 6 in. angles of the given thickness and distance back to back with the long leg against the web. For instance, suppose it is desired to find the moment of inertia of the flange angles of a plate girder having 2-8 in. x 6 in. x $\frac{3}{4}$ in. angles in each flange with a depth back to back of angles of $90\frac{1}{2}$ ins. For a depth of $90\frac{1}{2}$ ins. we find from the curve (Table 21) that the percentage to use is 85.9. The moment of inertia of the flange angles of a plate girder having 2-8 in. x 8 in. x $\frac{3}{4}$ in. flange angles $90\frac{1}{2}$ ins. back to back is 84759. The result sought is $84759 \times .859 = 72808$.

To obtain the moment of inertia of the net section of 8 in. x 6 in. angles with various combinations of rivets, multiply the moment of inertia of the gross section as found from Tables 16 and 21 by the proper percentage from Table 22.

Table 23 gives the moments of inertia, for both flanges, of cover plates of 10 ins. width and varying thicknesses and clear distances between plates. To obtain the moments of inertia of plates of widths different from 10 ins. for a given thickness of plates and distance apart, multiply by the ratio of gross or net width of plate sought to 10 ins. The result will be the gross or net moment of inertia of the desired plate for both flanges.

To find the moment of inertia of a plate 10 ins. wide and thicker than 1-9/16 ins. proceed as in the following example: Find the moment of inertia (I) of the coverplates in both flanges of a girder whose depth is $36\frac{1}{2}$ ins. between cover plates (back to back of angles). The cover plates are 10 ins. wide and total $2\frac{3}{4}$ ins. thick in each flange. The I for 10 in. plates $1\frac{1}{2}$ in. thick and $36\frac{1}{2}$ ins. apart is 10836. It now remains to find the I for cover plates 10 ins. wide, $1\frac{1}{4}$ ins. thick and of a distance apart equal to the out to out distance of the plates whose I has already been found. This distance is $39\frac{1}{2}$ ins. in this case. The I for 10 in. plate $1\frac{1}{4}$ ins. thick and $39\frac{1}{2}$ ins. apart is found to be 10385. The sum $10836 + 10385 = 21221$ is the I of cover plates 10 ins. wide, $2\frac{3}{4}$ ins. thick and $36\frac{1}{2}$ ins. apart. By extending this process it is evidently possible to compute the moment of inertia of any desired thickness of cover plates by adding a series of quantities properly chosen from the table. For widths other than 10 ins. compute first for 10 ins. and then use the multiplier which fits the case from Table 24.

Table 24 gives the multipliers for finding the moments of inertia of cover plates of various gross and net widths. The moment of inertia is first to be found from Table 23 for 10 ins. width and the proper thickness and distance apart. This quantity is then to be multiplied by the proper quantity from Table 24.

Tables 25 to 28 inclusive give the spacing of web stiffeners for various formulas. The shear per linear inch, used as an ordinate, is found by dividing the shear at any section by the effective

depth at that section, and will be recognized as a quantity which must be found when computing the pitch of flange rivets by the approximate method. Having found this shear follow it horizontally until it intersects the curve corresponding to the web thickness used. Go vertically downward from this intersection and the required distance between stiffeners may be read at once as an abscissa.

Tables 29 to 32 inclusive give shearing and bearing values of various sizes of rivets for various fibre stresses. The arrangement of these tables is a little different and it is believed more convenient than the arrangement found in most handbooks. The arrangement makes it especially easy to obtain the shearing or bearing stress in pounds per square inch for any given rivet and stress. This will be found helpful when the table is used to check over existing structures.

Tables 33 to 38 give various quantities which are useful to the draftsman. Their use is obvious.

Table 39 is a graphical diagram giving the allowable spacing for rivets of various diameters for different distances between gage lines. It is based on a distance of 3 diameters between centers of rivets. Knowing the distance between gage lines and the diameter of rivets the minimum spacing between rivets measured parallel to the gage lines may be read off at once.

Table 40 is a multiplication table for rivet spacing. Its use is obvious.

The use of the tables may be illustrated by designing the box girder of Chapter VI. The maximum moment is 25,935,000 inch-lbs. or 2,162,500 ft.-lbs. Assume the effective depth as 24 ins. and refer to Table 10. A line is drawn on the table through the required moment and the tensile fibre stress, 15,000 lb., to the point where it intersects the line marked area multiplied by depth. A line drawn through the effective depth and this intersection will cut the line of flange areas in the required area of the tension flange. It is found to be 72 sq. ins. The required area of the compression flange is the same in this case.

	Top flange.	Bottom flange
Required area	72.00	72.00
Web equivalent (Table 3) 24 in. $\times$ $1\frac{1}{4}$ in.	3.75	3.75
	68.25	68.25
Four $6 \times 6 \times \frac{1}{2}$ in. L's (Table 5)	23.00	19.00
	28)45.25	24)49.25
Total thickness of cover plates	1.62	2.05
Four $\frac{7}{16}$ in. plates	1.75	
One $\frac{9}{16}$ in. plate		0.56
Three $\frac{1}{2}$ in. plates		1.50
		2.06

Assuming four $\frac{1}{2}$ in. plates as was done in the text, we will find the moment of inertia of the gross section.

Webs aggregating $24 \times 1\frac{1}{4}$ (Table 11)		1440
Eight $6 \times 6 \times \frac{1}{2}$ L's, $24\frac{1}{2}$ ins. back to back, $2 \times$ value from Table 15		5300
Two $28 \times \frac{1}{2}$ in. pls. top and bottom, $24\frac{1}{2}$ ins. apart (Table 23).	3253	
Two $28 \times \frac{1}{2}$ in. pls. top and bottom, $26\frac{1}{2}$ ins. apart (Table 23).	3783	
	$7036 \times 2.8 =$	19700
		26440

The value of the moment of inertia of the net section is very readily found as follows—assuming that $\frac{7}{8}$ in. rivets are used throughout:

Web	$1440 \times \frac{3}{4}$ =	1080
Angles (Table 15)	$5300 \times .822$ =	4350
Cover plates (Table 24)	7036×2.4 =	16900
		22330

This is almost exactly the value found in the text but is obtained with much less labor.

If the moment of inertia is desired considering the gross section as available in the top flange and the net area as available in the bottom flange, it may be found approximately by averaging the two results above and will give

$$\begin{array}{r} 26440 \\ 22330 \\ \hline 2)48770 \\ \hline 24385 \end{array}$$

This result assumes that the neutral axis remains at the mid depth of the web which the author believes to be correct. (See text.)

TABLE 1

MOMENT DIAGRAM—COOPER'S E_{60} LOCOMOTIVE

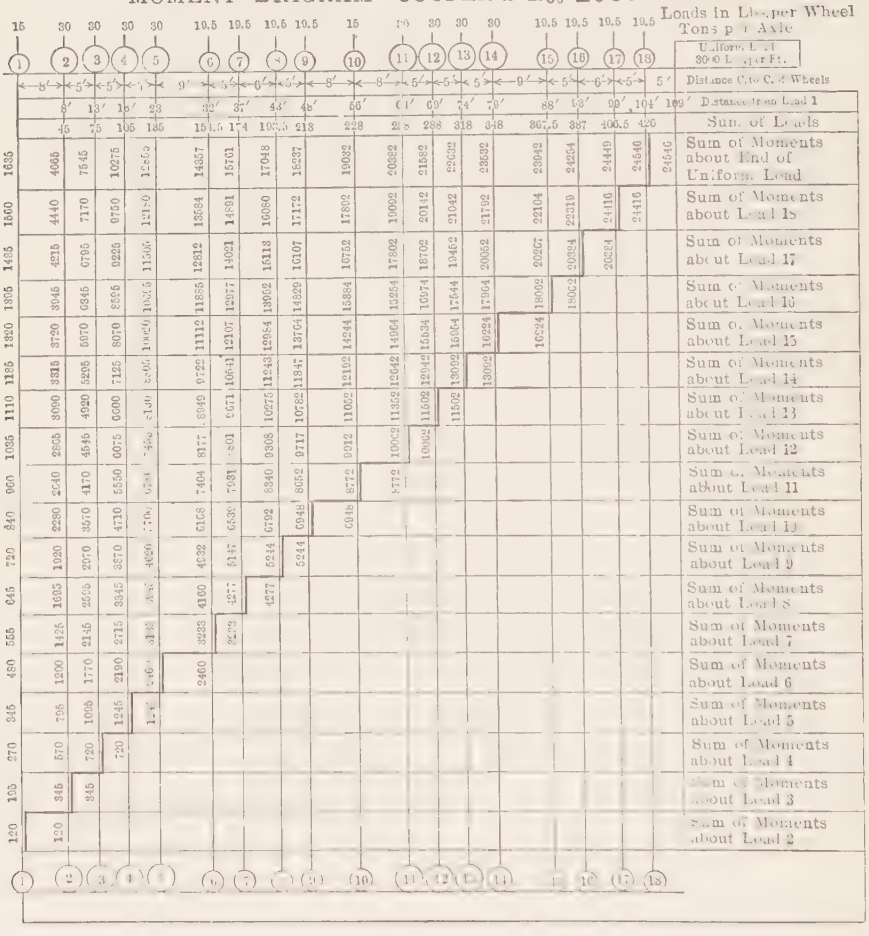

Loads in Lbs. per Wheel
Tons per Axle
Uniform Load 3000 Lbs. per Ft.
Distance C to C of Wheels
Distance from Load 1
Sum of Loads
Sum of Moments about End of Uniform Load
Sum of Moments about Load 18
Sum of Moments about Load 17
Sum of Moments about Load 16
Sum of Moments about Load 15
Sum of Moments about Load 14
Sum of Moments about Load 13
Sum of Moments about Load 12
Sum of Moments about Load 11
Sum of Moments about Load 10
Sum of Moments about Load 9
Sum of Moments about Load 8
Sum of Moments about Load 7
Sum of Moments about Load 6
Sum of Moments about Load 5
Sum of Moments about Load 4
Sum of Moments about Load 3
Sum of Moments about Load 2

TABLE 2. WEB AREAS

Web depth ins.	Web thickness.														
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
12	3 0	3 8	4 5	5 3	6 0	6.7	7.5	8 2	9.0	9 8	10 5	11.3	12 0
13	3 3	4 1	4 9	5 7	6 5	7.3	8.1	9 0	9.8	10.6	11 4	12.2	13 0
14	3 5	4 4	5 3	6.2	7 0	7.9	8.8	9.7	10.6	11.4	12.3	13.2	14.0
15	3 8	4 7	5 6	6 6	7 5	8.4	9.4	10.3	11.3	12.2	13.1	14.1	15 0
16	4 0	5 0	6 0	7 0	8 0	9.0	10.0	11.0	12.0	13 0	14 0	15.0	16 0
17	4 3	5 4	6.4	7 4	8 5	9.5	10.6	11.7	12.7	13.8	14 9	15.9	17.0
18	4 5	5 6	6.8	7 9	9 0	10.2	11.3	12.4	13.5	14 6	15.8	16.9	18.0
19	4 8	5 9	7 1	8 3	9 5	10.6	11.8	13.0	14.2	15.4	16.6	17.8	19 0
20	5 0	6 2	7 5	8.7	10 0	11.2	12.5	13 8	15.0	16 2	17 5	18.8	20.0
21	5 3	6 6	7.9	9.2	10.5	11.8	13.1	14.4	15.8	17.0	18.4	19.7	21 0
22	5 5	6 9	8 2	9.6	11.0	12.4	13.8	15.1	16.5	17.8	19.3	20.6	22 0
23	5 8	7 2	8.6	10.0	11 5	13.0	14.4	15 8	17.3	18.7	20.2	21.6	23.0
24	6 0	7 5	9.0	10.5	12.0	13.5	15.0	16.5	18.0	19.5	21.0	22.5	24.0
25	6 3	7 8	9 4	11.0	12 5	14.0	15.6	17 2	18.7	20.3	21.8	23.4	25.0
26	6 5	8 1	9 8	11.4	13 0	14.6	16.2	17 9	19.5	21.1	22.7	24.4	26 0
27	6 8	8 4	10 2	11.8	13 5	15.2	16.9	18 6	20.2	21.9	23.6	25.3	27 0
28	7 0	8.7	10 5	12.2	14 0	15.8	17.5	19.3	21.0	22.7	24.5	26.2	28 0
29	7 3	9.0	10 9	12.6	14 5	16.3	18.1	19 9	21.8	23.5	25.4	27.2	29 0
30	7 5	9 4	11 3	13 1	15.0	16.9	18.7	20.6	22.5	24.3	26.2	28.2	30 0
31	7 8	9 7	11 6	13 5	15 5	17.4	19.4	21 4	23.3	25.2	27.1	29.1	31 0
32	8 0	10 0	12 0	14 0	16 0	18.0	20.0	22 0	24.0	26 0	28.0	30.0	32.0
33	8 3	10 3	12.4	14 4	16 5	18.6	20.6	22.7	24.7	26.8	28.9	31.0	33.0
34	8 5	10 6	12.7	14.9	17.0	19.1	21.3	23.4	25.5	27.6	29 8	31.9	34 0
35	8 8	11 0	13.1	15 3	17.5	19.7	21.9	24.1	26.2	28.4	30.6	32.9	35 0
36	9.0	11.2	13 5	15.8	18 0	20.2	22.5	24.8	27.0	29 3	31.5	33.8	36 0
37	9.3	11.5	13 8	16.2	18.5	20.8	23.1	25 4	27.8	30 1	32.4	34.8	37.0
38	9.5	11.8	14 2	16.6	19 0	21.4	23.8	26 2	28.5	30.9	33.2	35.7	38 0
39	9 8	12 2	14.6	17.1	19 5	21.9	24.1	26 9	29.3	31.7	34.1	36.6	39 0
40	10.0	12 5	15 0	17.5	20.0	22.5	25.0	27 5	30.0	32.5	35.0	37.5	40 0
41	10.3	12.8	15 4	17.9	20.5	23.0	25.6	28 2	30.7	33.3	35.8	38.5	41 0
42	10 5	13 1	15 8	18.4	21.0	23.6	26.2	28 9	31.5	34.1	36 7	39.4	42.0
43	10 8	13 4	16.2	18.8	21.5	24.2	26.9	29.6	32.2	34 9	37.6	40.3	43 0
44	11 0	13.8	16.5	19.2	22.0	24.7	27.5	30 2	33.0	35 8	38 5	41.3	44.0
45	11 3	14 1	16 9	19 7	22 5	25.3	28.2	31 0	33.8	36 6	39.4	42.2	45.0
46	11 5	14 3	17.3	20 2	23 0	25 9	28.8	31.7	34.5	37.4	40.2	43.2	46 0
47	11.8	14.6	17.6	20.6	23 5	26 5	29.1	32.4	35.2	38 2	41.1	44.1	47 0
48	12 0	15.0	18.0	21.0	24 0	27 0	30.0	33.0	36.0	39.0	42.0	45.0	48 0
49	12.3	15.3	18.4	21.4	24 5	27.6	30.6	33.8	36.7	39.8	42.9	46.0	49.0
50	12 5	15 6	18.7	21.8	25.0	28.2	31.3	34.4	37.5	40 6	43 7	46.9	50 0
51	12 8	15.9	19 1	22.3	25 5	28 7	31.9	35.1	38.2	41 4	44 6	47.8	51 0
52	13 0	16 2	19.5	22 8	26.0	29 3	32.5	35.8	39.0	42 2	45.5	48.8	52 0
53	13 3	16.5	19.9	23 2	26 5	29.8	33.1	36 5	39.7	43 0	46 4	49.7	53 0
54	13.5	16.8	20.3	23.6	27 0	30.4	33.8	37.2	40.5	43.8	47.3	50.7	54.0

TABLE 2. WEB AREAS—(*Continued*)

Web depth ins.	Web thickness.												
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
55	13.8	17.2	20.6	24.0	27.5	31.0	34.4	37.8	41.3	44.7	48.2	51.6	55.0
56	14.0	17.5	21.0	24.5	28.0	31.5	35.0	38.5	42.0	45.5	49.0	52.5	56.0
57	14.2	17.8	21.4	25.0	28.5	32.1	35.6	39.2	42.7	46.3	49.9	53.4	57.0
58	14.5	18.1	21.8	25.4	29.0	32.6	36.2	39.9	43.5	47.1	50.8	54.4	58.0
59	14.7	18.4	22.1	25.8	29.5	33.2	36.9	40.6	44.2	47.9	51.6	55.3	59.0
60	15.0	18.7	22.5	26.2	30.0	33.8	37.5	41.2	45.0	48.8	52.5	56.3	60.0
61	15.3	19.0	22.9	26.7	30.5	34.3	38.2	41.9	45.7	49.6	53.4	57.2	61.0
62	15.5	19.4	23.3	27.1	31.0	34.9	38.8	42.6	46.5	50.4	54.2	58.1	62.0
63	15.8	19.7	23.7	27.5	31.5	35.4	39.4	43.3	47.3	51.2	55.1	59.1	63.0
64	16.0	20.0	24.0	28.0	32.0	36.0	40.0	44.0	48.0	52.0	56.0	60.0	64.0
65	16.2	20.3	24.3	28.4	32.5	36.6	40.6	44.7	48.8	52.8	56.9	61.0	65.0
66	16.5	20.6	24.7	28.9	33.0	37.1	41.2	45.4	49.5	53.7	57.8	61.9	66.0
67	16.7	21.0	25.1	29.3	33.5	37.7	41.9	46.1	50.2	54.5	58.6	62.8	67.0
68	17.0	21.3	25.5	29.8	34.0	38.2	42.6	46.8	51.0	55.3	59.5	63.8	68.0
69	17.3	21.6	25.9	30.2	34.5	38.8	43.2	47.5	51.8	56.1	60.4	64.7	69.0
70	17.5	21.9	26.2	30.6	35.0	39.4	43.8	48.2	52.5	56.9	61.2	65.6	70.0
71	17.8	22.2	26.6	31.0	35.5	40.0	44.5	48.8	53.2	57.7	62.1	66.6	71.0
72	18.0	22.5	27.0	31.5	36.0	40.5	45.0	49.5	54.0	58.5	63.0	67.5	72.0
73	18.2	22.8	27.4	31.9	36.5	41.0	45.6	50.2	54.8	59.4	63.9	68.5	73.0
74	18.5	23.1	27.8	32.4	37.0	41.6	46.2	50.9	55.5	60.2	64.7	69.4	74.0
75	18.7	23.4	28.2	32.8	37.5	42.2	46.9	51.6	56.2	61.0	65.6	70.4	75.0
76	19.0	23.8	28.5	33.2	38.0	42.8	47.5	52.3	57.0	61.8	66.5	71.3	76.0
77	19.3	24.1	28.9	33.7	38.5	43.4	48.2	53.0	57.8	62.6	67.4	72.2	77.0
78	19.5	24.4	29.3	34.1	39.0	43.9	48.8	53.6	58.5	63.4	68.2	73.2	78.0
79	19.8	24.7	29.6	34.6	39.5	44.4	49.4	54.3	59.2	64.2	69.1	74.1	79.0
80	20.0	25.0	30.0	35.0	40.0	45.0	50.0	55.0	60.0	65.0	70.0	75.0	80.0
81	20.2	25.3	30.4	35.4	40.5	45.6	50.6	55.7	60.8	65.8	70.9	76.0	81.0
82	20.5	25.6	30.7	35.8	41.0	46.1	51.3	56.4	61.5	66.6	71.7	76.9	82.0
83	20.7	25.9	31.1	36.3	41.5	46.7	51.9	57.1	62.2	67.5	72.6	77.9	83.0
84	21.0	26.2	31.5	36.8	42.0	47.2	52.5	57.8	63.0	68.3	73.5	78.8	84.0
85	21.3	26.6	31.8	37.2	42.5	47.8	53.1	58.4	63.8	69.1	74.4	79.7	85.0
86	21.5	26.9	32.2	37.6	43.0	48.4	53.8	59.1	64.5	69.9	75.2	80.6	86.0
87	21.8	27.2	32.6	38.0	43.5	48.9	54.4	59.8	65.2	70.7	76.1	81.5	87.0
88	22.0	27.5	33.0	38.5	44.0	49.5	55.0	60.5	66.0	71.6	77.0	82.4	88.0
89	22.2	27.8	33.4	38.9	44.5	50.0	55.6	61.2	66.8	72.4	77.9	83.4	89.0
90	22.5	28.2	33.8	39.4	45.0	50.6	56.3	61.9	67.5	73.2	78.8	84.4	90.0
91	22.7	28.5	34.1	39.8	45.5	51.2	56.9	62.6	68.2	74.0	79.6	85.3	91.0
92	23.0	28.8	34.5	40.2	46.0	51.8	57.5	63.3	69.0	74.8	80.5	86.3	92.0
93	23.3	29.1	34.9	40.7	46.5	52.3	58.2	64.0	69.8	75.6	81.3	87.2	93.0
94	23.5	29.4	35.2	41.1	47.0	52.9	58.8	64.7	70.5	76.4	82.2	88.1	94.0
95	23.8	29.7	35.6	41.5	47.5	53.5	59.5	65.3	71.2	77.2	83.1	89.0	95.0
96	24.0	30.0	36.0	42.0	48.0	54.0	60.0	66.0	72.0	78.0	84.0	90.0	96.0
97	24.2	30.3	36.4	42.4	48.5	54.6	60.6	66.7	72.8	78.8	84.8	90.9	97.0
98	24.5	30.6	36.8	42.8	49.0	55.1	61.2	67.4	73.5	79.7	85.7	91.8	98.0
99	24.7	30.9	37.2	43.3	49.5	55.7	61.9	68.1	74.2	80.5	86.6	92.8	99.0

TABLE 2. WEB AREAS—(*Continued*)

Web depth ins.	Web thickness.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
100	25.0	31.3	37.5	43.8	50.0	56.2	62.5	68.8	75.0	81.3	87.5	93.8	100.0
101	25.3	31.6	37.9	44.1	50.5	56.8	63.2	69.5	75.8	82.1	88.4	94.6	101.0
102	25.5	31.9	38.2	44.6	51.0	57.4	63.8	70.2	76.5	82.9	89.2	95.5	102.0
103	25.8	32.2	38.6	45.1	51.5	58.0	64.4	70.8	77.2	83.7	90.1	96.5	103.0
104	26.0	32.5	39.0	45.5	52.0	58.6	65.0	71.6	78.0	84.5	91.0	97.5	104.0
105	26.3	32.8	39.4	45.9	52.5	59.0	65.6	72.3	78.8	85.3	91.9	98.5	105.0
106	26.5	33.1	39.8	46.4	53.0	59.6	66.3	73.0	79.6	86.1	92.8	99.4	106.0
107	26.8	33.4	40.2	46.8	53.5	60.1	66.9	73.6	80.3	87.0	93.6	100.3	107.0
108	27.0	33.8	40.5	47.2	54.0	60.8	67.4	74.3	81.0	87.8	94.4	101.2	108.0
109	27.3	34.1	40.9	47.7	54.5	61.3	68.1	75.0	81.7	88.6	95.3	102.1	109.0
110	27.5	34.4	41.3	48.2	55.0	61.9	68.8	75.7	82.5	89 4	96.2	103.1	110.0
111	27.8	34.7	41.7	48.5	55.5	62.4	69.4	76.4	83.2	90.2	97.1	104.0	111.0
112	28.0	35.0	42.0	49.0	56.0	63.0	70.0	77.0	84.0	91.0	98.0	105.0	112.0
113	28.3	35.3	42.4	49.4	56.5	63.6	70.7	77.7	84.8	91.9	98.9	106.0	113.0
114	28.5	35.6	42.8	49.9	57.0	64.1	71.3	78.4	85.6	92.7	99.7	106.9	114.0
115	28.8	35.9	43.2	50.3	57.5	64.7	71.9	79.1	86.3	93.5	100.6	107.9	115.0
116	29.0	36.2	43.5	50.8	58.0	65.2	72.5	79.8	87.1	94.3	101.4	108.9	116.0
117	29 3	36 6	43 9	51 2	58 5	65 8	73 2	80 5	87 8	95.1	102.3	109.9	117.0
118	29.5	36.9	44.3	51.6	59.0	66.4	73.7	81.2	88.5	95.9	103.1	110.8	118.0
119	29.8	37.2	44.7	52.0	59.5	67.0	74.4	81.8	89.2	96.7	104.0	111.7	119.0
120	30.0	37.5	45.0	52.5	60.0	67.5	75.0	82.5	90.0	97.5	105.0	112.5	120.0

TABLE 3. WEB EQUIVALENTS $\frac{1}{8}$TH

Web depth ins.	Web thicknesses.														
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1		
12	38	47	.56	66	75	84	94	1.03	1 12	1 22	1 31	1 41	1 50		
13	41	.51	.61	.71	.81	.91	1 01	1.12	1 22	1 32	1 42	1 52	1 63		
14	44	.55	.66	.77	.88	.99	1 10	1.21	1 32	1 43	1 53	1 64	1 75		
15	47	.59	70	82	.94	1 05	1.17	1.29	1 41	1 52	1 64	1 76	1 88		
16	.50	.62	75	.87	1.00	1 12	1.25	1.37	1 50	1 62	1 75	1 87	2 00		
17	.53	.66	.80	.93	1.06	1.19	1 33	1.46	1.59	1 73	1 86	1 99	2 13		
18	.56	.70	85	.99	1.13	1 26	1 41	1.55	1 69	1 83	1 97	2 11	2 25		
19	.59	.74	.89	1.04	1.19	1 33	1 48	1.63	1.78	1.93	2 08	2 23	2 38		
20	.63	.78	.94	1.09	1.25	1 40	1 56	1.72	1 88	2 03	2 19	3 35	2 50		
21	.66	.82	99	1.15	1.31	1.47	1 64	1.80	1 97	2 13	2 30	2 46	2 63		
22	.69	.86	1 03	1.20	1.37	1 55	1 72	1.89	2 05	2 23	2 41	2 58	2 75		
23	.72	.90	1.08	1.25	1.44	1 62	1 80	1.98	2 16	2 34	2 52	2 70	2 88		
24	.75	.94	1.13	1.31	1.50	1 69	1 88	2.06	2 25	2 44	2 62	2 82	3 00		
25	.78	.97	1.17	1.37	1.56	1 75	1 95	2.15	2 34	2 54	2 73	2 93	3 13		
26	.81	1.01	1 22	1.42	1.62	1 82	2 03	2.24	2 44	2 64	2 84	3 05	3 25		
27	.84	1.05	1 27	1.48	1.69	1 90	2 11	2.32	2 53	2 74	2 95	3 17	3 38		
28	.88	1.09	1 31	1.53	1.75	1 97	2 19	2.41	2 62	2 84	3 06	3 28	3 50		
29	.91	1.13	1 36	1.57	1.81	2 04	2 26	2.49	2 72	2 94	3 17	3 40	3 63		
30	.94	1.17	1 41	1.64	1.87	2 11	2 34	2.58	2 81	3 04	3 28	3 52	3 75		
31	.97	1.21	1 45	1.69	1.94	2 18	2 42	2.67	2 91	3 15	3 39	3 64	3 88		
32	1 00	1.25	1 50	1.75	2.00	2 25	2 50	2.75	3 00	3 25	3 50	3 75	4 00		
33	1 03	1.29	1 55	1.80	2.06	2 32	2 58	2.84	3 09	3 35	3 61	3 87	4 13		
34	1 03	1.33	1 59	1.86	2.12	2 39	2 65	2.93	3 19	3 45	3 72	3 99	4 25		
35	1 10	1.37	1 64	1.91	2.19	2 46	2 74	3.01	3 28	3 55	3 83	4 11	4 38		
36	1 13	1.40	1 69	1.97	2.25	2 53	2 81	3.10	3 37	3 65	3 94	4 23	4 50		
37	1 16	1.44	1 73	2.02	2.31	2 60	2 89	3.18	3 47	3 76	4 05	4 35	4 63		
38	1 19	1.48	1 78	2.08	2.38	2 67	2 97	3.27	3 56	3 86	4 15	4 43	4 75		
39	1 22	1.52	1 83	2.14	2.44	2 74	3 05	3.35	3 66	3 96	4 25	4 58	4 88		
40	1 25	1.56	1 88	2.19	2.50	2 81	3 12	3.44	3 75	4 06	4 37	4 70	5 00		
41	1 28	1.60	1 92	2.24	2.56	2 88	3 20	3.53	3 84	4 16	4 48	4 82	5 13		
42	1 31	1.64	1 97	2.30	2.63	2 95	3 28	3.61	3 94	4 25	4 59	4 94	5 25		
43	1 34	1.68	2 02	2.35	2.69	3 02	3 36	3.70	4 03	4 35	4 70	5 05	5 38		
44	1 37	1.72	2 06	2.40	2.75	3 09	3 44	3.78	4 13	4 47	4 81	5 16	5 50		
45	1 41	1.76	2 11	2.46	2.81	3 16	3 52	3.87	4 22	4 57	4 92	5 28	5 63		
46	1 44	1.79	2 16	2.52	2.88	3 24	3 60	3.95	4 31	4 67	5 03	5 40	5 75		
47	1 47	1.83	2 20	2.57	2.94	3 31	3 68	4.05	4 40	4 77	5 14	5 52	5 88		
48	1 50	1.87	2 25	2.62	3.00	3 38	3 75	4.13	4 50	4 87	5 25	5 64	6 00		
49	1 53	1.91	2 30	2.68	3.06	3 45	3 83	4.22	4 59	4 98	5 36	5 75	6 13		
50	1 56	1.95	2 34	2 73	3.12	3 52	3 91	4 30	4.69	5.08	5.47	5 87	6 25		
51	1 59	1.99	2 39	2 79	3.19	3 59	3 99	4 39	4.78	5.18	5.58	5 98	6 38		
52	1 62	2.03	2 44	2 85	3.25	3 66	4 06	4 47	4.87	5.28	5.69	6 10	6 50		
53	1 65	2.06	2 49	2 90	3.31	3 73	4 14	4 56	4.96	5.38	5.80	6.22	6 63		
54	1 69	2.10	2 54	2 95	3.38	3 80	4 22	4 65	5.06	5.48	5.91	6.34	6.75		

TABLE 3. WEB EQUIVALENTS $\frac{1}{8}$TH—(*Continued*)

Web depths ins.	Web thicknesses.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
55	1.72	2.15	2.58	3.00	3.44	3.87	4.30	4.73	5.16	5.59	6.02	6.45	6.88
56	1.75	2.19	2.62	3.06	3.50	3.94	4.38	4.81	5.25	5.69	6.13	6.57	7.00
57	1.78	2.23	2.67	3.12	3.56	4.01	4.45	4.90	5.34	5.79	6.24	6.68	7.13
58	1.81	2.26	2.72	3.17	3.63	4.08	4.53	4.99	5.44	5.89	6.35	6.80	7.25
59	1.84	2.30	2.76	3.22	3.69	4.15	4.61	5.07	5.53	5.99	6.45	6.92	7.38
60	1.88	2.34	2.81	3.28	3.75	4.22	4.69	5.15	5.63	6.10	6.56	7.04	7.50
61	1.91	2.38	2.86	3.34	3.82	4.29	4.77	5.24	5.72	6.20	6.67	7.15	7.63
62	1.94	2.42	2.91	3.39	3.88	4.36	4.85	5.33	5.81	6.30	6.78	7.27	7.75
63	1.97	2.46	2.96	3.44	3.94	4.43	4.92	5.41	5.91	6.40	6.89	7.39	7.88
64	2.00	2.50	3.00	3.50	4.00	4.50	5.00	5.50	6.00	6.50	7.00	7.50	8.00
65	2.03	2.54	3.04	3.55	4.05	4.57	5.08	5.59	6.10	6.60	7.11	7.62	8.13
66	2.03	2.58	3.09	3.61	4.13	4.64	5.16	5.67	6.19	6.71	7.22	7.74	8.25
67	2.09	2.62	3.14	3.66	4.19	4.71	5.24	5.76	6.28	6.81	7.33	7.85	8.38
68	2.12	2.66	3.19	3.72	4.25	4.78	5.32	5.85	6.38	6.91	7.44	7.97	8.50
69	2.16	2.70	3.24	3.77	4.31	4.85	5.40	5.93	6.47	7.01	7.55	8.09	8.63
70	2.19	2.74	3.28	3.83	4.38	4.92	5.47	6.02	6.56	7 11	7.65	8.21	8.75
71	2.22	2.77	3.33	3.88	4.44	5.00	5.55	6.10	6.65	7.21	7.76	8.33	8.88
72	2.25	2.81	3.38	3.94	4.50	5.06	5.63	6.19	6.75	7.32	7.87	8.45	9.00
73	2.28	2.85	3.42	3.99	4.56	5.13	5.70	6.27	6.85	7.42	7.98	8.56	9.13
74	2.31	2.89	3.47	4.05	4.63	5.20	5.78	6.36	6.94	7.52	8.09	8.68	9.25
75	2.34	2.93	3.52	4.10	4.69	5.27	5.86	6.45	7.03	7.62	8.20	8.80	9.38
76	2.38	2.97	3.56	4.15	4.75	5.34	5.94	6.54	7.13	7.72	8.31	8.91	9.50
77	2.41	3.01	3.61	4.21	4.81	5.41	6.02	6.63	7.22	7.83	8.42	9.03	9.63
78	2.44	3.05	3.66	4.26	4.88	5.48	6.10	6.70	7.31	7.93	8.53	9.15	9.75
79	2.47	3.08	3.70	4.32	4.94	5.55	6.17	6.79	7.40	8.03	8.64	9.26	9.88
80	2.50	3.12	3.75	4.37	5.00	5.63	6.25	6.88	7.50	8.13	8.75	9.38	10.00
81	2.53	3.16	3.80	4.43	5.05	5.70	6.33	6.96	7.60	8.23	8.86	9.50	10.13
82	2.56	3.20	3.84	4.48	5.13	5.77	6.41	7.05	7.69	8.33	8.97	9.61	10.25
83	2.59	3.24	3.89	4.54	5.20	5.84	6.49	7.14	7.78	8.44	9.08	9.73	10.38
84	2.62	3.28	3.94	4.60	5.25	5.91	6.56	7.22	7.88	8.54	9.19	9.85	10.50
85	2.63	3.32	3.98	4.65	5.31	5.98	6.64	7.30	7.97	8.64	9.30	9.97	10.63
86	2.69	3.36	4.03	4.70	5.38	6.05	6.72	7.39	8.06	8.74	9.41	10.08	10.75
87	2.72	3.40	4.08	4.75	5.44	6.12	6.80	7.48	8.15	8.84	9.52	10.20	10.80
88	2.75	3.44	4.12	4.81	5.50	6.19	6.88	7.56	8.25	8.95	9.63	10.31	11.00
89	2.78	3.48	4.17	4.86	5.56	6.26	6.95	7.65	8.35	9.05	9.74	10.43	11.13
90	2.81	3.52	4.22	4.92	5.63	6.33	7.03	7.74	8.44	9.15	9.85	10.55	11.25
91	2.84	3.56	4.26	4.97	5.69	6.40	7.11	7.82	8.53	9.25	9.96	10.67	11.38
92	2.87	3.59	4.31	5.03	5.75	6.47	7.19	7.91	8.63	9.35	10.06	10.79	11.50
93	2.91	3.63	4.36	5.09	5.81	6.54	7.27	8.00	8.72	9.45	10.17	10.90	11.63
94	2.94	3.67	4.40	5.14	5.88	6.61	7.35	8.08	8.81	9.55	10.28	11.01	11.75
95	2.97	3.71	4.45	5.19	5.94	6.68	7.43	8.16	8.90	9.65	10.39	11.12	11.88
96	3.00	3.75	4.50	5.25	6.00	6.75	7.50	8.25	9.00	9.75	10.50	11.24	12.00
97	3.03	3.79	4.55	5.30	6.06	6.82	7.58	8.34	9.10	9.86	10.60	11.36	12.13
98	3.06	3.83	4.60	5.35	6.13	6.89	7.65	8.43	9.19	9.96	10.70	11.48	12.25
99	3.09	3.87	4.65	5.41	6.19	6.96	7.73	8.51	9.28	10.05	10.81	11.60	12.38

TABLE 3. WEB EQUIVALENTS $\frac{1}{8}$TH—(*Continued*)

Web depth ins.	Web thicknesses.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
100	3.13	3.91	4.69	5.47	6.25	7.03	7.81	8.60	9.38	10.16	10.93	11.71	12.50
101	3.16	3.95	4.74	5.52	6.31	7.10	7.90	8.69	9.47	10.26	11.04	11.82	12.63
102	3.19	3.98	4.78	5.58	6.37	7.18	7.97	8.77	9.56	10.37	11.15	11.95	12.75
103	3.22	4.02	4.83	5.64	6.44	7.25	8.05	8.86	9.65	10.47	11.26	12.08	12.88
104	3.25	4.06	4.88	5.69	6.50	7.32	8.13	8.95	9.75	10.57	11.38	12.20	13.00
105	3.28	4.10	4.92	5.74	6.56	7.38	8.21	9.03	9.85	10.67	11.49	12.30	13.13
106	3.32	4.14	4.97	5.80	6.63	7.45	8.29	9.12	9.95	10.77	11.60	12.41	13.25
107	3.35	4.18	5.02	5.85	6.69	7.52	8.35	9.20	10.04	10.88	11.70	12.54	13.38
108	3.38	4.22	5.07	5.90	6.75	7.59	8.44	9.29	10.12	10.98	11.80	12.66	13.50
109	3.41	4.26	5.11	5.96	6.81	7.66	8.52	9.38	10.21	11.08	11.91	12.78	13.63
110	3.44	4.30	5.16	6.02	6.88	7.73	8.60	9.46	10.30	11.18	12.02	12.90	13.75
111	3.47	4.34	5.21	6.07	3.94	7.80	8.68	9.55	10.40	11.28	12.13	13.00	13.88
112	3.50	4.38	5.25	6.13	7.00	7.87	8.76	9.63	10.50	11.38	12.25	13.12	14.00
113	3.54	4.42	5.30	6.18	7.06	7.94	8.84	9.72	10.60	11.49	12.36	13.24	14.13
114	3.57	4.45	5.35	6.24	7.13	8.01	8.91	9.80	10.70	11.59	12.47	13.36	14.25
115	3.60	4.49	5.40	6.29	7.19	8.08	8.99	9.89	10.79	11.69	12.58	13.48	14.38
116	3.63	4.53	5.44	6.35	7.25	8.15	9.07	9.98	10.89	11.79	12.69	13.60	14.50
117	3.66	4.57	5.49	6.40	7.32	8.23	9.15	10.06	10.98	11.89	12.80	13.72	14.63
118	3.69	4.61	5.54	6.45	7.38	8.30	9.22	10.14	11.07	11.99	12.90	13.83	14.75
119	3.72	4.65	5.59	6.50	7.44	8.37	9.30	10.22	11.16	12.09	13.01	13.95	14.88
120	3.75	4.69	5.63	6.56	7.50	8.44	9.38	10.31	11.25	12.20	13.12	14.08	15.00

TABLE 4. WEB EQUIVALENTS $\frac{1}{32}$TH

Web depth ins.	Web thickness.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
12	.25	.31	.37	.44	.50	.56	.63	.69	.75	.81	.87	.94	1.00
13	.27	.34	.41	.47	.54	.61	.68	.75	.81	.88	.95	1.02	1.08
14	.29	.37	.44	.51	.59	.66	.73	.81	.88	.95	1.03	1.10	1.16
15	.31	.39	.47	.55	.63	.70	.78	.86	.94	1.01	1.10	1.17	1.25
16	.33	.42	.50	.58	.67	.75	.83	.91	1.00	1.08	1.17	1.25	1.33
17	.35	.45	.53	.62	.71	.80	.89	.97	1.06	1.15	1.24	1.33	1.42
18	.37	.47	.56	.66	.75	.85	.94	1.03	1.13	1.22	1.31	1.41	1.50
19	.39	.49	.59	.69	.79	.89	.99	1.09	1.19	1.29	1.38	1.49	1.58
20	.42	.52	.63	.73	.83	.93	1.04	1.15	1.25	1.35	1.45	1.57	1.66
21	.44	.55	.66	.77	.87	.98	1.09	1.20	1.31	1.42	1.53	1.64	1.75
22	.46	.57	.69	.80	.91	1.03	1.15	1.26	1.37	1.49	1.61	1.72	1.83
23	.48	.60	.72	.83	.96	1.08	1.20	1.32	1.44	1.56	1.68	1.80	1.92
24	.50	.63	.75	.87	1.00	1.13	1.25	1.37	1.50	1.63	1.75	1.88	2.00
25	.52	.65	.78	.91	1.04	1.17	1.30	1.43	1.56	1.69	1.82	1.95	2.08
26	.54	.67	.81	.95	1.08	1.22	1.35	1.49	1.63	1.76	1.89	2.03	2.16
27	.56	.70	.84	.99	1.13	1.27	1.41	1.55	1.69	1.83	1.97	2.11	2.25
28	.59	.73	.87	1.02	1.17	1.31	1.46	1.61	1.75	1.89	2.04	2.19	2.33
29	.61	.75	.91	1.05	1.21	1.36	1.51	1.66	1.81	1.96	2.11	2.27	2.42
30	.63	.78	.94	1.09	1.25	1.41	1.56	1.72	1.87	2.03	2.19	2.35	2.50
31	.65	.81	.97	1.13	1.29	1.46	1.61	1.78	1.94	2.10	2.26	2.43	2.58
32	.67	.83	1.00	1.17	1.33	1.50	1.67	1.83	2.00	2.17	2.33	2.50	2.66
33	.69	.86	1.03	1.20	1.37	1.55	1.72	1.89	2.06	2.23	2.41	2.58	2.75
34	.71	.89	1.06	1.24	1.41	1.59	1.77	1.95	2.13	2.30	2.48	2.66	2.83
35	.73	.91	1.09	1.27	1.46	1.64	1.82	2.01	2.19	2.37	2.55	2.74	2.92
36	.75	.93	1.12	1.31	1.50	1.69	1.87	2.07	2.25	2.44	2.63	2.82	3.00
37	.77	.96	1.15	1.35	1.54	1.73	1.93	2.12	2.31	2.50	2.70	2.90	3.08
38	.79	.99	1.19	1.39	1.59	1.78	1.98	2.18	2.37	2.57	2.77	2.98	3.16
39	.81	1.01	1.22	1.43	1.63	1.83	2.03	2.24	2.44	2.64	2.84	3.06	3.25
40	.83	1.04	1.25	1.46	1.67	1.87	2.08	2.29	2.50	2.71	2.92	3.14	3.33
41	.85	1.07	1.28	1.49	1.71	1.92	2.14	2.35	2.56	2.78	2.99	3.22	3.42
42	.87	1.09	1.31	1.53	1.75	1.97	2.19	2.41	2.63	2.84	3.06	3.29	3.50
43	.89	1.12	1.34	1.57	1.79	2.02	2.24	2.47	2.69	2.91	3.13	3.37	3.58
44	.91	1.15	1.37	1.60	1.83	2.06	2.30	2.52	2.75	2.98	3.21	3.44	3.66
45	.94	1.17	1.41	1.64	1.87	2.11	2.35	2.58	2.81	3.05	3.28	3.52	3.75
46	.96	1.20	1.44	1.68	1.92	2.16	2.40	2.64	2.87	3.12	3.35	3.60	3.83
47	.98	1.22	1.47	1.71	1.96	2.21	2.45	2.70	2.93	3.18	3.42	3.68	3.92
48	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
49	1.02	1.28	1.53	1.79	2.04	2.30	2.56	2.81	3.06	3.32	3.58	3.83	4.08
50	1.04	1.30	1.56	1.82	2.08	2.35	2.61	2.86	3.12	3.39	3.65	3.91	4.16
51	1.06	1.33	1.59	1.86	2.13	2.39	2.66	2.92	3.18	3.46	3.72	3.98	4.25
52	1.08	1.36	1.63	1.90	2.17	2.44	2.71	2.98	3.25	3.52	3.79	4.06	4.33
53	1.10	1.38	1.66	1.93	2.21	2.49	2.76	3.04	3.31	3.59	3.86	4.14	4.42
54	1.13	1.41	1.69	1.97	2.25	2.54	2.82	3.10	3.38	3.66	3.94	4.22	4.50

TABLE 4. WEB EQUIVALENTS $\frac{1}{2}$TH—(*Continued*)

Web depth ins.	Web thickness.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
55	1.15	1.43	1.72	2.00	2.29	2.58	2.87	3.15	3.44	3.73	4.02	4.30	4.58
56	1.17	1.46	1.75	2 04	2.33	2.62	2.92	3.21	3.50	3.79	4.09	4.38	4.66
57	1.19	1.49	1.78	2.08	2.37	2.67	2.97	3.27	3.56	3.86	4.16	4.45	4.75
58	1.21	1.51	1.81	2.11	2.42	2.72	3.02	3.32	3.62	3.92	4.23	4.53	4.83
59	1.23	1.53	1.84	2.15	2.46	2.77	3.07	3.38	3.68	3.99	4.30	4.61	4.92
60	1.25	1.56	1.87	2.19	2.50	2.82	3.12	3.43	3.75	4.06	4.37	4.69	5.00
61	1.27	1.59	1.91	2.23	2.54	2.86	3.18	3.49	3.81	4.13	4.45	4.76	5.08
62	1.29	1.61	1.94	2.26	2.58	2.91	3.23	3.55	3.87	4.20	4.52	4.84	5.16
63	1.31	1.64	1.97	2.29	2.62	2.95	3.28	3.61	3.94	4.26	4.59	4.92	5.25
64	1.33	1.67	2.00	2.33	2.66	3.00	3.33	3.66	4.00	4.33	4.66	5.00	5.33
65	1.35	1.69	2.03	2.36	2.71	3.05	3.38	3.72	4.07	4.40	4.74	5.08	5.42
66	1.37	1.72	2.06	2.40	2.75	3.09	3.43	3.78	4.13	4.47	4.81	5.16	5.50
67	1.39	1.75	2.09	2.44	2.79	3.14	3.49	3.84	4.19	4.54	4.88	5.24	5.58
68	1.41	1.77	2.13	2.48	2.83	3.19	3.55	3.90	4.25	4.61	4.96	5.31	5.66
69	1.44	1.80	2.16	2.51	2.88	3.24	3.60	3.95	4.31	4.67	5.03	5.39	5.75
70	1.46	1.83	2.19	2.55	2.92	3.28	3.65	4.01	4.37	4.74	5.10	5.47	5.83
71	1.48	1.85	2.22	2.58	2.96	3.33	3.70	4.07	4.43	4.81	5.17	5.55	5.92
72	1.50	1.87	2.25	2.62	3.00	3.38	3.75	4.12	4.50	4.88	5.25	5.63	6.00
73	1.52	1.90	2.28	2.66	3.04	3.42	3.80	4.18	4.56	4.95	5.32	5.70	6.08
74	1.54	1.93	2.31	2.70	3.08	3.47	3.85	4.24	4.63	5.01	5.39	5.78	6.16
75	1.56	1.95	2.34	2.74	3.12	3.52	3.91	4.30	4.69	5.08	5.46	5.86	6.25
76	1.59	1.98	2.37	2.77	3.17	3.57	3.96	4.36	4.75	5.15	5.54	5.94	6.33
77	1.61	2.01	2.41	2.81	3.21	3.61	4.01	4.42	4.81	5.22	5.61	6.02	6.42
78	1.63	2.04	2.44	2 84	3.26	3.66	4.06	4.47	4.87	5.28	5.68	6.10	6.50
79	1.65	2.06	2.47	2.88	3.30	3.70	4.11	4.52	4.93	5.35	5.75	6.17	6.58
80	1.67	2.08	2.50	2.92	3.34	3.75	4.17	4.58	5.00	5.42	5.83	6.25	6.66
81	1.69	2.11	2.53	2.96	3.38	3.80	4.22	4.64	5.06	5.49	5.90	6.33	6.75
82	1.71	2.14	2.56	2.99	3.42	3.85	4.27	4.70	5.12	5.55	5.97	6.40	6.83
83	1.73	2.16	2.59	3.03	3.46	3.89	4.32	4.76	5.19	5.62	6.05	6.48	6.92
84	1.75	2.19	2.62	3.06	3.50	3.94	4.38	4.81	5.25	5.69	6.12	6.56	7.00
85	1.77	2.21	2.65	3.10	3.54	3.98	4.43	4.86	5.31	5.75	6.20	6.64	7.08
86	1.79	2.24	2.68	3.14	3.58	4.03	4.48	4.92	5.37	5.82	6.27	6.72	7.16
87	1.81	2.27	2.72	3.17	3.62	4.07	4.53	4.98	5.44	5.89	6.34	6.80	7 25
88	1.83	2.29	2.75	3.21	3.66	4.12	4.58	5.04	5.50	5.96	6.41	6.87	7.33
89	1.85	2.32	2.78	3.24	3.71	4.17	4.63	5.10	5.56	6.03	5.49	6.95	7.42
90	1 87	2 35	2 81	3 28	3 75	1 22	4 68	5 15	5 62	6.10	6.56	7.03	7.50
91	1.89	2.37	2.84	3.32	3.79	4.27	4.74	5.21	5.68	6.16	6.63	7.11	7.58
92	1.91	2.39	2.87	3.35	3.83	4.31	4.79	5.27	5.75	6.23	6.70	7.19	7.66
93	1.94	2.42	2.91	3.39	3.87	4.35	4.85	5.33	5.81	6.30	6.77	7.27	7.75
94	1.96	2.44	2.94	3.42	3.92	4.40	4.90	5.39	5.87	6.37	6.84	7.35	7.83
95	1.98	2.47	2.97	3.46	3.96	4.45	4.95	5.45	5.94	6.44	6.92	7.42	7.92
96	2.00	2.50	3.00	3.50	4.00	4.50	5.00	5.50	6.00	6.50	7.00	7.50	8.00
97	2.02	2.52	3.03	3.54	4.04	4.55	5.05	5.55	6.06	6.57	7.06	7.57	8.08
98	2.04	2 55	3.06	3.57	4.08	4.60	5.10	5.61	6.12	6.64	7.13	7.65	8.16
99	2.06	2.58	3.10	3.61	4.12	4.65	5.15	5.67	6.19	6.71	7.20	7.73	8.25

TABLE 4. WEB EQUIVALENTS $\frac{1}{32}$TH—(*Continued*)

Web depth ins.	Web thickness.												
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
100	2.08	2.61	3.13	3 65	4.16	4.69	5.20	5.73	6 25	6.77	7.28	7 81	8.33
101	2.11	2.63	3.16	3 68	4.21	4.74	5.26	5.79	6 31	6.83	7.36	7 88	8.42
102	2.13	2.65	3.19	3 72	4.25	4.79	5.31	5.85	6 37	6.90	7.43	7.96	8.50
103	2.15	2.68	3.22	3 76	4.29	4.83	5.36	5.90	6 43	6.97	7.50	8.04	8.58
104	2.17	2.71	3.25	3 79	4.33	4.88	5.41	5.96	6 50	7.04	7.58	8.12	8.66
105	2.19	2.74	3.28	3 83	4.37	4.92	5.46	6.02	6 57	7.10	7.65	8.20	8 75
106	2.21	2.76	3.31	3 87	4.41	4.96	5.52	6.08	6 64	7.17	7.73	8 28	8.83
107	2.23	2.79	3.34	3 90	4.45	5.01	5.57	6.13	6 70	7.24	7.80	8.36	8.92
108	2.25	2.82	3.38	3 93	4.50	5.06	5.62	6.19	6.75	7.31	7.87	8.44	9.00
109	2.27	2.84	3.41	3 97	4.54	5.10	5.67	6.25	6 80	7.38	7.95	8.52	9.08
110	2.29	2.87	3.44	4 01	4.58	5.15	5.73	6.30	6 86	7.45	8.02	8.60	9 16
111	2.32	2.89	3.47	4 05	4.62	5.20	5.79	6.36	6 93	7.51	8.09	8.67	9.25
112	2.34	2.92	3.50	4 08	4.66	5.25	5.84	6.42	7 00	7.58	8.16	8.75	9 33
113	2.36	2.95	3.53	4 12	4.70	5.30	5.89	6.48	7 07	7.65	8.23	8 83	9 42
114	2.38	2.97	3.57	4 15	4.75	5.34	5.94	6.54	7.14	7.72	8.30	8.90	9.50
115	2.40	2.99	3.60	4 19	4.79	5.39	5.99	6.59	7.20	7.79	8.38	8.98	9.58
116	2.42	3.02	3.63	4 23	4.83	5.44	6.04	6.65	7.26	7.86	8.45	9.06	9.66
117	2.44	3.05	3.66	4 26	4.88	5.48	6.10	6.71	7 32	7.93	8.53	9.14	9 75
118	2.46	3.08	3.69	4 30	4.92	5.53	6.15	6.76	7.38	7.99	8.60	9.22	9 83
119	2.48	3.10	3.72	4 34	4.96	5.58	6.20	6.82	7.44	8.06	8.67	9.30	9 92
120	2.50	3.13	3.75	4 37	5.00	5.62	6.25	6.88	7.50	8.13	8.75	9.38	10 00

TABLE 5. GROSS AND NET AREAS OF 2 ANGLES—EQUAL LEG ANGLES

Size	Thickness	Weight of 2 angles	Gross Area of 2 angles	Net area of 2 angles			
				¾″ Rivets		⅞″ Rivets	
				(2)	(4)	(2)	(4)
2½″ x 2½″	3/16	6.2	1.82	1.5	1.2	...	...
	1/4	8.2	2.38	1.9	1.5	...	...
	5/16	10.0	2.94	2.4	1.8	...	...
	3/8	11.8	3.48	2.8	2.1	...	...
	7/16	13.6	4.00	3.2	2.4	...	...
	1/2	15.4	4.50	3.6	2.7	...	...
	9/16	17.0	5.00	4.0	3.0	...	...
3″ x 3″	1/4	9.8	2.88	2.4	2.0	2.2	1.9
	5/16	12.2	3.56	3.0	2.5	2.9	2.3
	3/8	14.4	4.22	3.6	2.9	3.5	2.7
	7/16	16.6	4.88	4.1	3.3	4.0	3.1
	1/2	18.8	5.50	4.6	3.7	4.5	3.5
	9/16	20.8	6.12	5.1	4.2	5.0	3.9
	5/8	23.0	6.72	5.6	4.5	5.5	4.2
	11/16	25.0	7.32	6.1	4.9	5.9	4.6
3½″ x 3½″	5/16	14.4	4.18	3.6	3.1	3.6	2.9
	3/8	17.0	4.98	4.3	3.6	4.2	3.5
	7/16	19.6	5.76	5.0	4.2	4.9	4.0
	1/2	22.2	6.50	5.6	4.7	5.5	4.5
	9/16	24.8	7.26	6.3	5.3	6.1	5.0
	5/8	27.2	7.98	6.9	5.8	6.7	5.5
	11/16	29.6	8.68	7.5	6.3	7.3	5.9
	3/4	32.0	9.38	8.1	6.8	7.9	6.4
	13/16	34.2	10.06	8.6	7.2	8.4	6.8
	7/8	36.6	10.72	9.2	7.7	9.0	7.2
4″ x 4″	5/16	16.4	4.82	4.3	3.7	4.2	3.6
	3/8	19.6	5.72	5.1	4.4	5.0	4.2
	7/16	22.6	6.62	5.9	5.1	5.7	4.9
	1/2	25.6	7.50	6.6	5.7	6.5	5.5
	9/16	28.6	8.38	7.4	6.4	7.2	6.1
	5/8	31.4	9.22	8.1	7.0	8.0	6.7
	11/16	34.2	10.06	8.9	7.7	8.7	7.3
	3/4	37.0	10.88	9.6	8.3	9.4	7.9
	13/16	39.8	11.68	10.3	8.8	10.1	8.4
	7/8	42.4	12.48	11.0	9.4	10.7	9.0
6″ x 6″	3/8	29.8	8.72	8.0	7.2	7.9	7.0
	7/16	34.4	10.12	9.2	8.4	9.1	8.2
	1/2	39.2	11.50	10.5	9.5	10.4	9.3
	9/16	43.8	12.88	11.7	10.6	11.6	10.4
	5/8	48.4	14.22	13.0	11.7	12.8	11.4
	11/16	53.0	15.56	14.2	12.8	14.0	12.5
	3/4	57.4	16.88	15.4	13.9	15.2	13.5
	13/16	62.0	18.18	16.6	14.9	16.4	14.5
	7/8	66.2	19.48	17.7	16.0	17.5	15.6
	15/16	70.6	20.76	18.9	17.0	18.7	16.6
	1	74.8	22.00	20.0	18.0	19.7	17.5

TABLE 5. GROSS AND NET AREAS OF 2 ANGLES—EQUAL LEG ANGLES
—(*Continued*)

Size	Thickness	Weight of 2 angles	Gross Area of 2 angles	Net area of 2 angles			
				$\frac{7}{8}$″ Rivets		1″ Rivets	
				(2)	(4)	(2)	(4)
8″ x 8″	$\frac{1}{2}$	52.8	15.50	14.5	13.5	14.4	13.3
	$\frac{9}{16}$	59.2	17.38	16.2	15.1	16.1	14.9
	$\frac{5}{8}$	65.4	19.22	18.0	16.7	17.8	16.4
	$\frac{11}{16}$	71.6	21.06	19 7	18.3	19.5	18.0
	$\frac{3}{4}$	77.8	22.88	21.4	19.9	21.2	19.5
	$\frac{13}{16}$	84.0	24.68	23.1	21.4	22.9	21.0
	$\frac{7}{8}$	90.0	26.48	24.7	23.0	24.5	22.6
	$\frac{15}{16}$	96.2	28.26	26.4	24.5	26.2	24.1
	1	102.0	30.00	28.0	26.0	27.7	25.5
	$1\frac{1}{16}$	108.0	31.76	29.6	27.5	29.4	27.0
	$1\frac{1}{8}$	113.8	33.48	31.2	29.0	30.9	28.4

TABLE 6. GROSS AND NET AREAS OF 2 ANGLES—UNEQUAL LEG ANGLES

Size	Thickness	Weight of 2 angles	Gross Area of 2 angles	Net Area of 2 angles			
				$\frac{7}{8}$″ Rivets		$\frac{7}{8}$″ Rivets	
				(2)	(4)	(2)	(4)
3″ x 2½″	$\frac{1}{4}$	9.0	2.64	2.2	1.7	.....	.....
	$\frac{3}{16}$	11.2	3.26	2.7	2.1	.....	.....
	$\frac{3}{8}$	13.2	3.86	3.2	2.5	.....	.....
	$\frac{7}{16}$	15.2	4.44	3.7	2.9	.....	.....
	$\frac{1}{2}$	17.0	5.00	4.1	3.2	.....	.....
	$\frac{9}{16}$	19.0	5.56	4.6	3.6	.....	.....
	$\frac{5}{8}$	20.8	6.10	5.0	3.9	.....	.....
3½″ x 2½″	$\frac{1}{4}$	9.8	2.88	2.4	2.0	2.2	1.9
	$\frac{5}{16}$	12.2	3.56	3.0	2.5	2.9	2.3
	$\frac{3}{8}$	14.4	4.22	3.6	2.9	3.5	2.7
	$\frac{7}{16}$	16.6	4.88	4.1	3.3	4.0	.....
	$\frac{1}{2}$	18.8	5.50	4.6	3.7	4.5	.....
	$\frac{9}{16}$	20.8	6.12	5.1	4.2	5.0	.....
	$\frac{5}{8}$	23.0	6.72	5.6	4.5	5.5	.....
	$\frac{11}{16}$	25.0	7.32	6.1	4.9	5.9	.....
	$\frac{3}{4}$	26.8	7.88	6.6	5.3	6.4	.....
3½″ x 3″	$\frac{5}{16}$	13.2	3.88	3.3	2.8	3.3	2.6
	$\frac{3}{8}$	15.8	4.60	3.9	3.3	3.9	3.1
	$\frac{7}{16}$	18.2	5.32	4.6	3.8	4.4	3.6
	$\frac{1}{2}$	20.4	6.00	5.1	4.2	5.0	4.0
	$\frac{9}{16}$	22.8	6.68	5.7	4.7	5.6	4.4
	$\frac{5}{8}$	25.0	7.36	6.3	5.2	6.1	4.9
	$\frac{11}{16}$	27.2	8.00	6.8	5.6	6.6	5.2
	$\frac{3}{4}$	29.4	8.64	7.3	6.0	7.1	.....
	$\frac{13}{16}$	31.6	9.26	7.8	6.4	7.6	.....
	$\frac{7}{8}$	33.6	9.86	8.4	6.9	8.1	.....
4″ x 3″	$\frac{5}{16}$	14.4	4.18	3.6	3.1	3.6	2.9
	$\frac{3}{8}$	17.0	4.98	4.3	3.6	1.2	3.5
	$\frac{7}{16}$	19.6	5.76	5.0	4.2	4.9	4.0
	$\frac{1}{2}$	22.2	6.50	5.6	4.7	5.5	4.5
	$\frac{9}{16}$	24.8	7.26	6.3	5.3	6.1	5.0
	$\frac{5}{8}$	27.2	7.98	6.9	5.8	6.7	5.5
	$\frac{11}{16}$	29.6	8.68	7.5	6.3	7.3	5.9
	$\frac{3}{4}$	32.0	9.38	8.1	6.8	7.9	6.4
	$\frac{13}{16}$	34.2	10.06	8.6	7.2	8.4	6.8
	$\frac{7}{8}$	36.6	10.72	9.2	7.7	9.0	7.2
5″ x 3″	$\frac{5}{16}$	16.4	4.82	4.3	3.7	4.2	3.6
	$\frac{3}{8}$	19.6	5.72	5.1	4.4	5.0	4.2
	$\frac{7}{16}$	22.6	6.62	5.9	5.1	5.7	4.9
	$\frac{1}{2}$	25.6	7.50	6.6	5.7	6.5	5.5
	$\frac{9}{16}$	28.6	8.38	7.4	6.4	7.2	6.1
	$\frac{5}{8}$	31.4	9.22	8.1	7.0	8.0	6.7
	$\frac{11}{16}$	34.2	10.06	8.9	7.7	8.7	7.3
	$\frac{3}{4}$	37.0	10.88	9.6	8.3	9.4	7.9
	$\frac{13}{16}$	39.8	11.68	10.3	8.8	10.1	8.4
	$\frac{7}{8}$	42.4	12.48	11.0	9.4	10.7	9.0

TABLE 6. GROSS AND NET AREAS OF 2 ANGLES—UNEQUAL LEG ANGLES —(*Continued*)

Size	Thickness	Weight of 2 angles	Gross Area of 2 angles	Net area of 2 angles			
				$\frac{3}{4}$″ Rivets		$\frac{7}{8}$″ Rivets	
				(2)	(4)	(2)	(4)
5″ x 3½″	$\frac{5}{16}$	17.4	5.12	4.6	4.0	4.5	3.9
	$\frac{3}{8}$	20.8	6.10	5.4	4.8	5.3	4.6
	$\frac{7}{16}$	24.0	7.06	6.3	5.5	6.2	5.3
	$\frac{1}{2}$	27.2	8.00	7.1	6.2	7.0	6.0
	$\frac{9}{16}$	30.4	8.94	8.0	7.0	7.8	6.7
	$\frac{5}{8}$	33.6	9.86	8.7	7.7	8.6	7.3
	$\frac{11}{16}$	36.6	10.76	9.5	8.3	9.4	8.0
	$\frac{3}{4}$	39.6	11.64	10.3	9.0	10.1	8.6
	$\frac{13}{16}$	42.6	12.50	11.1	9.7	10.9	9.2
	$\frac{7}{8}$	45.4	13.36	11.8	10.3	11.6	9.8
	$\frac{15}{16}$	48.4	14.18	12.5	10.9	12.3	10.4
6″ x 3½″	$\frac{3}{8}$	23.4	6.86	6.2	5.5	6.1	5.4
	$\frac{7}{16}$	27.0	7.94	7.2	6.4	7.1	6.2
	$\frac{1}{2}$	30.6	9.00	8.1	7.2	8.0	7.0
	$\frac{9}{16}$	34.2	10.06	9.1	8.1	8.9	7.8
	$\frac{5}{8}$	37.8	11.10	10.0	8.9	9.9	8.6
	$\frac{11}{16}$	41.2	12.12	10.9	9.7	10.7	9.4
	$\frac{3}{4}$	44.8	13.14	11.8	10.5	11.6	10.1
	$\frac{13}{16}$	48.0	14.12	12.7	11.3	12.5	10.9
	$\frac{7}{8}$	51.4	15.10	13.6	12.0	13.4	11.6
	$\frac{15}{16}$	54.6	16.06	14.4	12.8	14.2	12.3
	1	57.8	17.00	15.3	13.5	15.0	13.0
6″ x 4″	$\frac{3}{8}$	24.6	7.22	6.6	5.9	6.5	5.7
	$\frac{7}{16}$	28.6	8.38	7.6	6.8	7.5	6.6
	$\frac{1}{2}$	32.4	9.50	8.6	7.7	8.5	7.5
	$\frac{9}{16}$	36.2	10.62	9.6	8.7	9.5	8.4
	$\frac{5}{8}$	40.0	11.72	10.6	9.5	10.5	9.2
	$\frac{11}{16}$	43.6	12.82	11.6	10.4	11.4	10.1
	$\frac{3}{4}$	47.2	13.88	12.6	11.3	12.4	10.9
	$\frac{13}{16}$	50.8	14.94	13.5	12.1	13.3	11.7
	$\frac{7}{8}$	54.4	15.98	14.4	12.9	14.2	12.5
	$\frac{15}{16}$	57.8	17.00	15.4	13.7	15.1	13.2
	1	61.2	18.00	16.2	14.5	16.0	14.0
8″ x 6″	$\frac{1}{2}$	46.0	13.50	12.5	11.5	12.4	11.3
	$\frac{9}{16}$	51.6	15.12	14.0	12.9	13.9	12.6
	$\frac{5}{8}$	57.0	16.72	15.5	14.2	15.3	13.9
	$\frac{11}{16}$	62.4	18.30	16.9	15.5	16.8	15.2
	$\frac{3}{4}$	67.8	19.88	18.4	16.9	18.2	16.5
	$\frac{13}{16}$	73.0	21.42	19.8	18.2	19.6	17.8
	$\frac{7}{8}$	78.2	22.96	21.2	19.4	21.0	19.0
	$\frac{15}{16}$	83.4	24.50	22.6	20.7	22.4	20.3
	1	88.6	26.00	24.0	22.0	23.7	21.5

Thickness in inches and sixteenths	Widths in inches										
	8	10	12	13	14	15	16	18	20	22	24
5	2.50	3.13	3.75	4.06	4.38	4.69	5.00	5.63	6.25	6.88	7.50
6	3.00	3.75	4.50	4.88	5.25	5.63	6.00	6.75	7.50	8.25	9.00
7	3.50	4.38	5.25	5.69	6.13	6.56	7.00	7.88	8.75	9.63	10.50
½ 8	4.00	5.00	6.00	6.50	7.00	7.50	8.00	9.00	10.00	11.00	12.00
9	4.50	5.63	6.75	7.31	7.88	8.44	9.00	10.13	11.25	12.38	13.50
10	5.00	6.25	7.50	8.13	8.75	9.38	10.00	11.25	12.50	13.75	15.00
11	5.50	6.88	8.25	8.94	9.63	10.31	11.00	12.38	13.75	15.13	16.50
¾ 12	6.00	7.50	9.00	9.75	10.50	11.25	12.00	13.50	15.00	16.50	18.00
13	6.50	8.13	9.75	10.56	11.38	12.19	13.00	14.63	16.25	17.88	19.50
14	7.00	8.75	10.50	11.38	12.25	13.13	14.00	15.75	17.50	19.25	21.00
15	7.50	9.38	11.25	12.19	13.13	14.06	15.00	16.88	18.75	20.63	22.50
1 inch	8.00	10.00	12.00	13.00	14.00	15.00	16.00	18.00	20.00	22.00	24.00
1	8.50	10.63	12.75	13.81	14.88	15.94	17.00	19.13	21.25	23.38	25.50
2	9.00	11.25	13.50	14.63	15.75	16.88	18.00	20.25	22.50	24.75	27.00
3	9.50	11.88	14.25	15.44	16.63	17.81	19.00	21.38	23.75	26.13	28.50
¼ 4	10.00	12.50	15.00	16.25	17.50	18.75	20.00	22.50	25.00	27.50	30.00
5	10.50	13.13	15.75	17 06	18 38	19 69	21 00	23 63	26 25	28 88	31 50
6	11.00	13.75	16.50	17 88	19 25	20 63	22 00	24 75	27 50	30 25	33 00
7	11.50	14.38	17.25	18 69	20 13	21 56	23 00	25 88	28 75	31 63	34 50
½ 8	12.00	15.00	18.00	19 50	21 00	22 50	24 00	27 00	30 00	33 00	36 00
9	12.50	15.63	18.75	20 31	21 88	23 44	25 00	28 13	31 25	34 38	37 50
10	13.00	16.25	19.50	21 13	22 75	24 38	26 00	29 25	32 50	35 75	39 00
11	13.50	16.88	20 25	21 94	23 63	25 31	27 00	30 38	33 75	37 13	40 50
¾ 12	14.00	17.50	21 00	22 75	24 50	26 25	28 00	31 50	35 00	38 50	42 00
13	14.50	18.13	21 75	23 56	25 38	27 19	29 00	32 63	36 25	39 88	43 50
14	15.00	18.75	22 50	24 38	26 25	28 13	30 00	33 75	37 50	41 25	45 00
15	15.50	19.38	23 25	25 19	27 13	29 06	31 00	34 88	38 75	42 63	46 50
2 inches	16.00	20.00	24 00	26 00	28 00	30 00	32 00	36 00	40 00	44 00	48 00
1	16.50	20.63	24.75	26 81	28 88	30 94	33 00	37 13	41 25	45 38	49 50
2	17.00	21.25	25.50	27 63	29 75	31 88	34 00	38 25	42 50	46 75	51 00
3	17.50	21.88	26.25	28 44	30 63	32 81	35 00	39 38	43 75	48 13	52 50
¼ 4	18.00	22.50	27.00	29 25	31 50	33 75	36 00	40 50	45 00	49 50	54 00
5	18.50	23.13	27 75	30 06	32 38	34 69	37 00	41 63	46 25	50 88	55 50
6	19.00	23.75	28 50	30 88	33 25	35 63	38 00	42 75	47 50	52 25	57 00
7	19.50	24.38	29 25	31 69	34 13	36 56	39 00	43 88	48 75	53 63	58 50
½ 8	20.00	25.00	30 00	32 50	35 00	37 50	40 00	45 00	50 00	55 00	60 00
9	20.50	25.63	30 75	33 31	35 88	38 44	41 00	46 13	51 25	56 38	61 50
10	21.00	26.25	31 50	34 13	36 75	39 38	42 00	47 25	52 50	57 75	63 00
11	21.50	26.88	32.25	34 94	37 63	40 31	43 00	48 38	53 75	59 13	64 50
¾ 12	22.00	27.50	33 00	35 75	38 50	41 25	44 00	49 50	55 00	60 50	66 00
13	22.50	28.13	33.75	36 56	39 38	42 19	45 00	50 63	56 25	61 88	67 50
14	23.00	28.75	34 50	37 38	40 25	43 13	46 00	51 75	57 50	63 25	69 00
15	23.50	29.38	35.25	38 19	41 13	44 06	47 00	52 88	58 75	64 63	70 50
3 inches	24.00	30.00	36.00	39 00	42 00	45 00	48 00	54 00	60 00	66 00	72 00
1	24.50	30.63	36.75	39 81	42 88	45 94	49 00	55 13	61 25	67 38	73 50
2	25.00	31.25	37.50	40.63	43 75	46 88	50 00	56 25	62 50	68 75	75 00
3	25.50	31.88	38.25	41.44	44 63	47 81	51 00	57 38	63 75	70 13	76 50
¼ 4	26.00	32.50	39.00	42 25	45 50	48 75	52 00	58 50	65 00	71 50	78 00
5	26.50	33.13	39 75	43 06	46 38	49 69	53 00	59 63	66 25	72 88	79 50
6	27.00	33.75	40.50	43 88	47 25	50 63	54 00	60 75	67 50	74 25	81 00
7	27.50	34.38	41.25	44 69	48 13	51 56	55 00	61 88	68 75	75 63	82 50
½ 8	28.00	35.00	42.00	45 50	49 00	52 50	56 00	63 00	70 00	77 00	84 00
9	28.50	35.63	42.75	46 31	49 88	53 44	57 00	64 13	71 25	78 38	85 50
10	29.00	36.25	43.50	47 13	50 75	54 38	58 00	65 25	72 50	79 75	87 00
11	29.50	36.88	44.25	47 94	51 63	55 31	59 00	66 38	73 75	81 13	88 50
¾ 12	30.00	37.50	45.00	48 75	52 50	56 25	60 00	67 50	75 00	82 50	90 00
13	30.50	38.13	45.75	49 56	53 38	57 19	61 00	68 63	76 25	83 88	91 50
14	31.00	38.75	46.50	50 38	54 25	58 13	62 00	69 75	77 50	85 25	93 00
15	31.50	39.38	47.25	51 19	55 13	59 06	63 00	70 88	78 75	86 63	94 50
4 inches	32.00	40.00	48.00	52 00	56 00	60 00	64 00	72 00	80 00	88 00	96 00

TABLE 7. GROSS AREA OF PLATES—(*Continued*)

Thickness in inches and sixteenths	Widths in inches										
	8	10	12	13	14	15	16	18	20	22	24
	32.50	40.63	48.75	52.81	56.88	60.94	65.00	73.13	81.25	89.38	97.50
	33.00	41.25	49.50	53.63	57.75	61.88	66.00	74.25	82.50	90.75	99.00
	33.50	41.88	50.25	54.44	58.63	62.81	67.00	75.38	83.75	92.13	100.50
¼	34.00	42.50	51.00	55.25	59.50	63.75	68.00	76.50	85.00	93.50	102.00
	34.50	43.13	51.75	56.06	60.38	64.69	69.00	77.63	86.25	94.88	103.50
	35.00	43.75	52.50	56.88	61.25	65.63	70.00	78.75	87.50	96.25	105.00
	35.50	44.38	53.25	57.69	62.13	66.56	71.00	79.88	88.75	97.63	106.50
½	36.00	45.00	54.00	58.50	63.00	67.50	72.00	81.00	90.00	99.00	108.00
	36.50	45.63	54.75	59.31	63.88	68.44	73.00	82.13	91.25	100.38	109.50
	37.00	46.25	55.50	60.13	64.75	69.38	74.00	83.25	92.50	101.75	111.00
	37.50	46.88	56.25	60.94	65.63	70.31	75.00	84.38	93.75	103.13	112.50
¾	38.00	47.50	57.00	61.75	66.50	71.25	76.00	85.50	95.00	104.50	114.00
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	105.88	115.50
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	107.25	117.00
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	108.63	118.50
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	100.00	110.00	120.00

TABLE 8. NET AREAS OF PLATES—TWO $\frac{7}{8}$″ RIVETS ALLOWED FOR

Thickness in inches and sixteenths	Gross width in inches										
	8	10	12	13	14	15	16	18	20	22	24
5	1.88	2.50	3.13	3.44	3.75	4.06	4.38	5.00	5.63	6.25	6.88
6	2.25	3.00	3.75	4.13	4.50	4.88	5.25	6.00	6.75	7.50	8.25
7	2.63	3.50	4.38	4.81	5.25	5.69	6.13	7.00	7.88	8.75	9.63
½ 8	3.00	4.00	5.00	5.50	6.00	6.50	7.00	8.00	9.00	10.00	11.00
9	3.38	4.50	5.63	6.19	6.75	7.31	7.88	9.00	10.13	11.25	12.38
10	3.75	5.00	6.25	6.88	7.50	8.13	8.75	10.00	11.25	12.50	13.75
11	4.13	5.50	6.88	7.56	8.25	8.94	9.63	11.00	12.38	13.75	15.13
¾ 12	4.50	6.00	7 50	8.25	9.00	9.75	10.50	12.00	13.50	15.00	16.50
13	4.88	6.50	8.13	8.94	9.75	10.56	11.38	13.00	14.63	16.25	17.88
14	5.25	7.00	8.75	9.63	10.50	11.38	12.25	14.00	15.75	17.50	19.25
15	5.63	7.50	9.38	10.31	11.25	12.19	13.13	15.00	16 88	18.75	20.63
1 inch 0	6.00	8.00	10.00	11.00	12.00	13.00	14.00	16.00	18.00	20.00	22.00
1	6.38	8.50	10.63	11.69	12.75	13.81	14.88	17.00	19 13	21 25	23 38
2	6.75	9.00	11.25	12.38	13.50	14.63	15.75	18.00	20 2	22 50	24 75
3	7.13	9.50	11.88	13.06	14.25	15.44	16.63	19.00	21 38	23 75	26 13
¼ 4	7.50	10.00	12.50	13.75	15.00	16.25	17.50	20 00	22 50	25 00	27 50
5	7.88	10.50	13.13	14.44	15.75	17.06	18.38	21.00	23 63	26 25	28 88
6	8.25	11.00	13.75	15.13	16.50	17.88	19.25	22.00	24 75	27 50	30 25
7	8.63	11.50	14.38	15.81	17.25	18.69	20.13	23.00	25 88	28 75	31 63
½ 8	9.00	12.00	15.00	16.50	18.00	19.50	21.00	24 00	27 00	30 00	33 00
9	9.38	12.50	15.63	17.19	18.75	20.31	21.88	25.00	28 13	31 25	34 38
10	9.75	13.00	16.25	17.88	19.50	21.13	22.75	26.00	29 25	32 50	35 75
11	10.13	13.50	16.88	18.56	20.25	21.94	23.63	27.00	30 38	33 75	37 13
¾ 12	10.50	14.00	17.50	19.25	21.00	22.75	24.50	28.00	31 50	35 00	38 50
13	10.88	14 50	18 13	19 94	21 75	23 56	25 38	29 00	32 63	36 25	39 88
14	11.25	15 00	18 75	20 63	22 50	24 38	26 25	30 00	33 75	37 50	41 25
15	11.63	15 50	19 38	21 31	23 25	25 19	27 13	31 00	34 88	38 75	42 63
2 inches 0	12.00	16 00	20 00	22 00	24 00	26 00	28 00	32 00	36 00	40 00	44 00
1	12.38	16 50	20 63	22 69	24 75	26 81	28 88	33 00	37 13	41 25	45 38
2	12.75	17 00	21 25	23 38	25 50	27 63	29 75	34 00	38 25	42 50	46 75
3	13 13	17 50	21 88	24 06	26 25	28 44	30 63	35 00	39 38	43 75	48 13
¼ 4	13.50	18 00	22 50	24 75	27 00	29 25	31 50	36 00	40 50	45 00	49 50
5	13.88	18 50	23 13	25 44	27 75	30 06	32 38	37 00	41 63	46 25	50 88
6	14.25	19 00	23 75	26 13	28 50	30 88	33 25	38 00	42 75	47 50	52 25
7	14.63	19 50	24 38	26 81	29 25	31 69	34 13	39 00	43 88	48 75	53 63
½ 8	15.00	20 00	25 00	27 50	30 00	32 50	35 00	40 00	45 00	50 00	55 00
9	15.38	20 50	25 63	28 19	30.75	33 31	35 88	41 00	46 13	51 25	56 38
10	15.75	21 00	26 25	28 88	31 50	34 13	36 75	42 00	47 25	52 50	57 75
11	16.13	21 50	26 88	29 56	32 25	34 94	37 63	43 00	48 38	53 75	59 13
¾ 12	16.50	22 00	27 50	30 25	33 00	35 75	38 50	44 00	49 50	55 00	60 50
13	16.88	22 50	28 13	30 94	33 75	36 56	39 38	45 00	50 63	56 25	61 88
14	17.25	23 00	28 75	31 63	34 50	37 38	40 25	46 00	51 75	57 50	63 25
15	17.63	23 50	29 38	32 31	35 25	38 19	41 13	47 00	52 88	58 75	64 63
3 inches 0	18.00	24 00	30.00	33 00	36 00	39 00	42 00	48 00	54 00	60 00	66 00
1	18.38	24.50	30.63	33.69	36.75	39.81	42.88	49.00	55.13	61.25	67.38
2	18.75	25.00	31.25	34.38	37.50	40.63	43.75	50.00	56.25	62.50	68.75
3	19.13	25.50	31.88	35.06	38.25	41.44	44.63	51.00	57.38	63.75	70.13
¼ 4	19.50	26.00	32.50	35.75	39.00	42.25	45.50	52.00	58.50	65.00	71.50
5	19.88	26.50	33.13	36.44	39.75	43.06	46.38	53.00	59.63	66.25	72.88
6	20.25	27.00	33.75	37.13	40.50	43.88	47.25	54.00	60.75	67.50	74.25
7	20.63	27.50	34.38	37.81	41.25	44.69	48.13	55.00	61.88	68.75	75.63
½ 8	21.00	28.00	35.00	38.50	42.00	45.50	49.00	56.00	63.00	70.00	77.00
9	21.38	28.50	35.63	39.19	42.75	46.31	49.88	57.00	64.13	71.25	78.38
10	21.75	29.00	36.25	39.88	43.50	47.13	50.75	58.00	65.25	72.50	79.75
11	22.13	29.50	36.88	40.56	44.25	47.94	51.63	59.00	66.38	73.75	81.13
¾ 12	22.50	30.00	37.50	41.25	45.00	48.75	52.50	60.00	67.50	75.00	82.50
13	22.88	30.50	38.13	41.94	45.75	49 56	53.38	61.00	68.63	76.25	83.88
14	23.25	31.00	38.75	42.63	46.50	50.38	54.25	62.00	69.75	77.50	85.25
15	23.63	31.50	39.38	43.31	47.25	51.19	55.13	63.00	70.88	78.75	86.63
4 inches 0	24.00	32.00	40.00	44.00	48.00	52.00	56.00	64.00	72.00	80.00	88.00

TABLE 8. NET AREAS OF PLATES—TWO $\frac{7}{8}$" RIVETS ALLOWED FOR—(*Continued*)

Thickness in inches and sixteenths	Gross width in inches										
	8	10	12	13	14	15	16	18	20	22	24
1	24.38	32.50	40.63	44.69	48.75	52.81	56.88	65.00	73.13	81.25	89.38
2	24.75	33.00	41.25	45.38	49.50	53.63	57.75	66.00	74.25	82.50	90.75
3	25.13	33.50	41.88	46.06	50.25	54.44	58.63	67.00	75.38	83.75	92.13
$\frac{1}{4}$ 4	25.50	34.00	42.50	46.75	51.00	55.25	59.50	68.00	76.50	85.00	93.50
5	25.88	34.50	43.13	47.44	51.75	56.06	60.38	69.00	77.63	86.25	94.88
6	26.25	35.00	43.75	48.13	52.50	56.88	61.25	70.00	78.75	87.50	96.25
7	26.63	35.50	44.38	48.81	53.25	57.69	62.13	71.00	79.88	88.75	97.63
$\frac{1}{2}$ 8	27.00	36.00	45.00	49.50	54.00	58.50	63.00	72.00	81.00	90.00	99.00
9	27.38	36.50	45.63	50.19	54.75	59.31	63.88	73.00	82.13	91.25	100.38
10	27.75	37.00	46.25	50.88	55.50	60.13	64.75	74.00	83.25	92.50	101.75
11	28.13	37.50	46.88	51.56	56.25	60.94	65.63	75.00	84.38	93.75	103.13
$\frac{3}{4}$ 12	28.50	38.00	47.50	52.25	57.00	61.75	66.50	76.00	85.50	95.00	104.50
13	28.88	38.50	48.13	52.94	57.75	62.56	67.38	77.00	86.63	96.25	105.88
14	29.25	39.00	48.75	53.63	58.50	63.38	68.25	78.00	87.75	97.50	107.25
15	29.63	39.50	49.38	54.31	59.25	64.19	69.13	79.00	88.88	98.75	108.63
5 inches 0	30.00	40.00	50.00	55.00	60.00	65.00	70.00	80.00	90.00	100.00	110.00

TABLE 9. NET AREAS OF PLATES FOR VARIOUS GROSS WIDTHS IN INCHES

Thickness in inches and sixteenths	Two $\frac{3}{4}''$ Rivets out			Two 1″ Rivets out				
	8	10	12	16	18	20	22	24
5	1.96	2.58	3.20	4.30	4.92	5.55	6.17	6.80
6	2.35	3.10	3.84	5.16	5.90	6.66	7.40	8.15
7	2.74	3.62	4.48	6.02	6.89	7.77	8.64	9.51
$\frac{1}{2}$ 8	3.13	4.13	5.13	6.88	7.88	8.88	9.88	10.87
9	3.52	4.64	5.77	7.74	8.86	9.99	11.11	12.23
10	3.91	5.16	6.41	8.60	9.85	11.10	12.34	13.59
11	4.30	5.67	7.05	9.46	10.83	12.21	13.58	14.95
$\frac{3}{4}$ 12	4.69	6.19	7.69	10.32	11.82	13.32	14.81	16.31
13	5.08	6.71	8.33	11.18	12.80	14.43	16.05	17.67
14	5.48	7.23	8.97	12.04	13.79	15.54	17.28	19.03
15	5.87	7.74	9.61	12.80	14.77	16.65	18.52	20.39
1 inch 0	6.25	8.25	10.25	13.75	15.75	17.75	19.75	21.75
1	6.65	8.77	10.89	14.61	16.74	18.86	20.99	23.11
2	7.04	9.28	11.53	15.47	17.72	19.97	22.22	24.47
3	7.42	9.80	12.17	16.33	18.71	21.08	23.46	25.83
$\frac{1}{4}$ 4	7.82	10.31	12.81	17.19	19.69	22.19	24.69	27.19
5	8.21	10.83	13.45	18.05	20.68	23.30	25.93	28.55
6	8.60	11.34	14.09	18.90	21.66	24.41	27.16	29.91
7	9.00	11.86	14.73	19.76	22.64	25.52	28.39	31.27
$\frac{1}{2}$ 8	9.39	12.38	15.38	20.63	23.63	26.63	29.63	32.63
9	9.78	12.89	16.02	21.48	24.61	27.74	30.86	33.98
10	10.15	13.41	16.66	22.34	25.59	28.85	32.10	35.34
11	10.55	13.92	17.30	23.20	26.58	29.96	33.33	36.70
$\frac{3}{4}$ 12	10.94	14.44	17.94	24.06	27.57	31.07	34.57	38.06
13	11.33	14.95	18.58	24.92	28.55	32.18	35.80	39.42
14	11.72	15.47	19.22	25.78	29.53	33.29	37.03	40.78
15	12.11	15.98	19.86	26.64	30.52	34.40	38.27	42.14
2 inches 0	12.50	16.50	20.50	27.50	31.50	35.50	39.50	43.50
1	12.89	17.01	21.14	28.36	32.49	36.61	40.74	44.86
2	13.29	17.53	21.78	29.22	33.47	37.72	41.97	46.22
3	13.68	18.04	22.42	30.08	34.46	38.83	43.21	47.58
$\frac{1}{4}$ 4	14.07	18.56	23.05	30.94	35.44	39.94	44.44	48.94
5	14.46	19.07	23.60	31.80	36.43	41.05	45.68	50.30
6	14.85	19.59	24.34	32.66	37.41	42.16	46.91	51.66
7	15.23	20.10	24.99	33.52	38.39	43.27	48.14	53.02
$\frac{1}{2}$ 8	15.63	20.63	25.63	34.38	39.38	44.38	49.38	54.38
9	16.02	21.14	26.27	35.23	40.36	45.49	50.61	55.73
10	16.42	21.66	26.91	36.09	41.35	46.60	51.85	57.09
11	16.81	22.17	27.55	36.95	42.33	47.71	53.08	58.45
$\frac{3}{4}$ 12	17.21	22.69	28.19	37.81	43.32	48.82	54.32	59.81
13	17.59	23.20	28.83	38.67	44.30	49.93	55.55	61.17
14	17.98	23.72	29.47	39.53	45.28	51.04	56.78	62.53
15	18.36	24.23	30.11	40.39	46.27	52.15	58.02	63.89
3 inches 0	18.75	24.75	30.75	41.25	47.25	53.25	59.25	65.25

TABLE 9. NET AREAS OF PLATES FOR VARIOUS GROSS WIDTHS IN INCHES —(*Continued*)

Thickness in inches and sixteenths	Two $\frac{3}{4}$″ Rivets out			Two 1″ Rivets out				
	8	10	12	16	18	20	22	24
1	19.15	25.26	31.39	42.11	48.24	54.36	60.49	66.61
2	19.54	25.78	32.03	42.97	49.22	55.47	61.72	67.97
3	19.94	26.29	32.67	43.83	50.21	55.58	62.96	69.33
$\frac{1}{4}$ 4	20.32	26.81	33.31	44.69	51.19	57.69	64.19	70.69
5	20.70	27.33	33.95	45.55	52.18	58.80	65.43	72.05
6	21.10	27.84	34.59	46.41	53.16	59.91	66.66	73.41
7	21.49	28.35	35.24	47.27	54.14	61.02	67.89	74.77
$\frac{1}{2}$ 8	21.88	28.88	35.88	48.13	55.13	62.13	69.13	76.13
9	22.27	29.39	36.51	48.99	56.11	63.24	70.36	77.48
10	22.66	29.91	37.16	49.84	57.10	64.35	71.60	78.84
11	23.05	30.42	37.80	50.70	58.08	65.46	72.83	80.20
$\frac{3}{4}$ 12	23.44	30.94	38.44	51.56	59.07	66.57	74.07	81.56
13	23.83	31.45	39.08	52.42	60.05	67.68	75.30	82.92
14	24.22	31.97	39.72	53.28	61.03	68.79	76.53	84.28
15	24.61	32.48	40.36	54.14	62.02	69.90	77.77	85.64
4 inches 0	25.00	33.00	41.00	55.00	63.00	71.00	79.00	87.00
1	25.39	33.52	41.64	55.86	63.99	72.11	80.24	88.36
2	25.78	34.03	42.28	56.72	64.97	73.22	81.47	89.72
3	26.17	34.55	42.92	57.58	65.96	74.33	82.71	91.08
$\frac{1}{4}$ 4	26.56	35.06	43.56	58.44	66.94	75.44	83.94	92.44
5	26.95	35.38	44.20	59.30	67.93	76.55	85.18	93.80
6	27.34	36.09	44.84	60.16	68.91	77.66	86.41	95.16
7	27.73	36.61	45.48	61.02	69.89	78.77	87.64	96.52
$\frac{1}{2}$ 8	28.13	37.13	46.13	61.88	70.88	79.88	88.88	97.98
9	28.52	37.64	46.77	62.74	71.86	80.99	90.11	99.23
10	28.91	38.16	47.41	63.59	72.85	82.10	91.35	100.59
11	29.30	38.67	48.05	64.45	73.83	83.21	92.58	101.95
$\frac{3}{4}$ 12	29.69	39.19	48.69	65.31	74.82	84.32	93.82	103.31
13	30.08	39.70	49.33	66.17	75.80	85.43	95.05	104.67
14	30.47	40.22	49.97	67.03	76.78	86.54	96.28	106.03
15	30.86	40.73	50.61	67.89	77.77	87.65	97.52	107.39
5 inches 0	31.25	41.25	51.25	68.75	78.75	88.75	98.75	108.75

TABLE 10

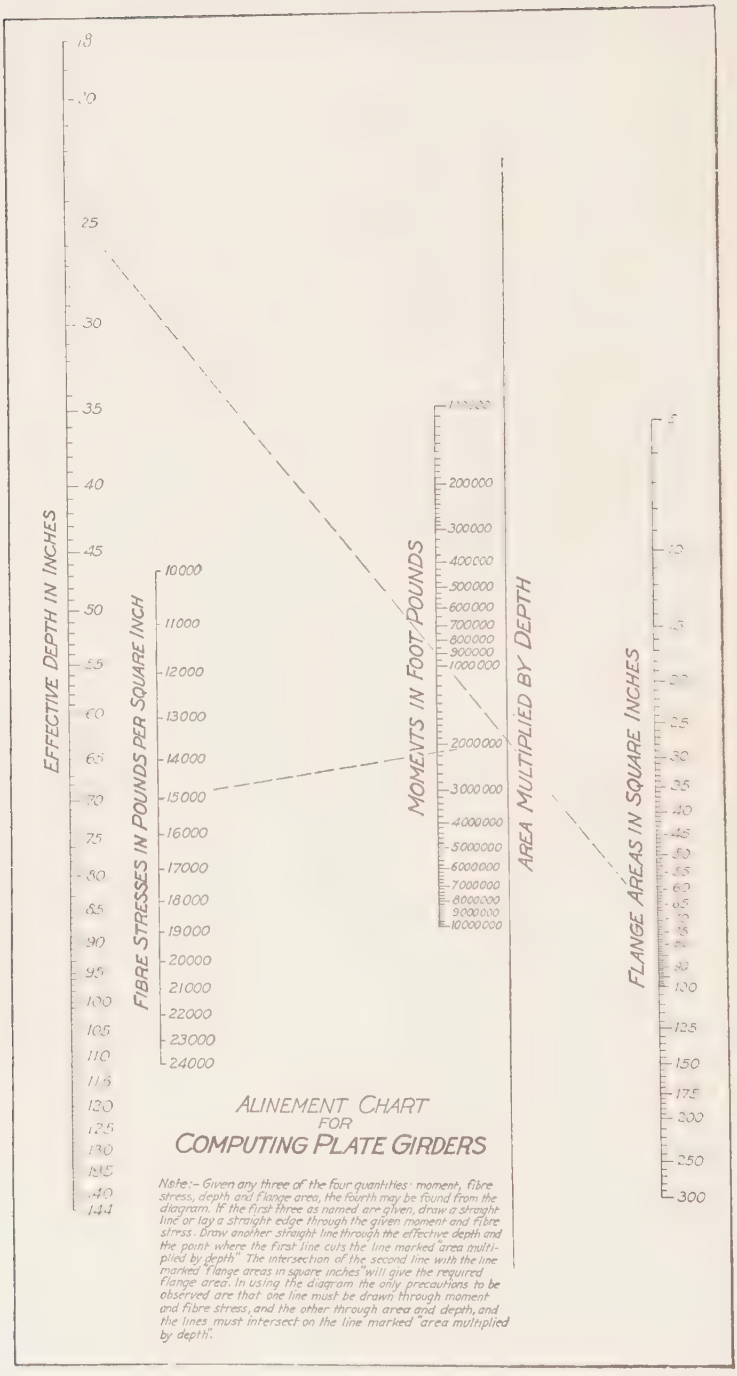

EFFECTIVE DEPTH IN INCHES
18
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
105
110
115
120
125
130
135
140
144
FIBRE STRESSES IN POUNDS PER SQUARE INCH
10000
11000
12000
13000
14000
15000
16000
17000
18000
19000
20000
21000
22000
23000
24000
MOMENTS IN FOOT-POUNDS
200000
300000
400000
500000
600000
700000
800000
900000
1000000
2000000
3000000
4000000
5000000
6000000
7000000
8000000
9000000
10000000
AREA MULTIPLIED BY DEPTH
FLANGE AREAS IN SQUARE INCHES
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
90
100
125
150
175
200
250
300
ALINEMENT CHART
FOR
COMPUTING PLATE GIRDERS
Note:- Given any three of the four quantities: moment, fibre stress, depth and flange area, the fourth may be found from the diagram. If the first three as named are given, draw a straight line or lay a straight edge through the given moment and fibre stress. Draw another straight line through the effective depth and the point where the first line cuts the line marked "area multiplied by depth". The intersection of the second line with the line marked "flange areas in square inches" will give the required flange area. In using the diagram the only precautions to be observed are that one line must be drawn through moment and fibre stress, and the other through area and depth, and the lines must intersect on the line marked "area multiplied by depth".

TABLE 11. MOMENTS OF INERTIA OF WEBS

Web depth ins.	Web thickness.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
12	36	45	54	63	72	81	90	99	108	117	126	135	144
13	46	57	69	80	92	103	114	126	137	149	160	172	183
14	57	71	86	100	114	129	143	157	172	186	200	214	229
15	70	88	105	123	141	158	176	193	211	229	246	264	281
16	85	107	128	149	171	192	213	235	256	277	299	320	341
17	102	128	154	179	205	230	256	281	307	333	358	384	409
18	122	152	182	213	243	273	304	334	365	395	425	456	486
19	143	179	214	250	286	322	357	393	429	464	500	536	572
20	167	208	250	292	333	375	417	458	500	542	583	625	667
21	193	241	289	338	386	434	482	531	579	627	675	724	772
22	222	277	333	388	444	499	555	610	666	721	776	832	887
23	253	317	380	444	507	570	634	697	760	824	887	951	1014
24	288	360	432	504	576	648	720	792	864	936	1008	1080	1152
25	326	407	488	570	651	732	814	895	977	1058	1139	1221	1302
26	366	458	549	641	732	824	915	1007	1099	1190	1282	1373	1465
27	410	513	615	718	820	923	1025	1128	1230	1333	1435	1538	1640
28	457	572	686	800	915	1029	1143	1258	1372	1486	1601	1715	1829
29	508	635	762	889	1016	1143	1270	1397	1524	1651	1778	1905	2032
30	563	703	844	984	1125	1266	1406	1547	1688	1828	1969	2109	2250
31	621	776	931	1086	1240	1397	1551	1708	1861	2018	2172	2325	2480
32	683	853	1024	1195	1365	1536	1707	1877	2048	2219	2389	2560	2731
33	750	938	1125	1312	1500	1688	1875	2063	2250	2438	2625	2813	3000
34	819	1024	1228	1433	1638	1842	2047	2252	2457	2661	2866	3071	3275
35	893	1117	1340	1565	1786	2010	2234	2455	2680	2905	3130	3346	3572
36	972	1215	1458	1701	1944	2187	2430	2673	2916	3159	3402	3645	3888
37	1057	1320	1585	1850	2114	2376	2640	2904	3170	3435	3700	3964	4228
38	1143	1429	1715	2000	2286	2572	2858	3144	3430	3715	4001	4287	4573
39	1238	1546	1854	2162	2470	2780	3090	3400	3710	4020	4330	4640	4952
40	1333	1667	2000	2333	2667	3000	3333	3667	4000	4333	4667	5000	5333
41	1437	1795	2154	2514	2874	3230	3590	3950	4310	4670	5030	5390	5748
42	1544	1929	2315	2701	3087	3473	3859	4245	4631	5016	5402	5788	6174
43	1658	2070	2485	2900	3316	3728	4140	4555	4970	5385	5800	6215	6632
44	1775	2218	2662	3106	3549	3993	4437	4880	5324	5768	6211	6655	7099
45	1900	2375	2850	3325	3800	4275	4750	5225	5700	6175	6650	7125	7600
46	2028	2535	3042	3549	4056	4563	5070	5577	6084	6590	7097	7604	8111
47	2163	2705	3244	3785	4326	4865	5410	5950	6488	7025	7570	8110	8652
48	2304	2880	3456	4032	4608	5184	5760	6336	6912	7488	8064	8640	9216
49	2450	3062	3676	4290	4900	5510	6124	6738	7352	7956	8580	9190	9800
50	2604	3255	3906	4557	5208	5859	6510	7161	7813	8464	9115	9766	10417
51	2763	3455	4145	4836	5527	6218	6910	7590	8290	8975	9680	10370	11054
52	2929	3662	4394	5126	5859	6591	7323	8056	8788	9520	10253	10985	11717
53	3100	3875	4650	5425	6200	6976	7752	8525	9300	10075	10850	11625	12400
54	3281	4101	4921	5741	6561	7381	8201	9021	9842	10662	11482	12302	13122
55	3467	4333	5200	6066	6934	7800	8667	9534	10400	11268	12134	13000	13868
56	3658	4573	5488	6400	7320	8232	9147	10061	10976	11890	12805	13720	14635
57	3858	4822	5786	6751	7716	8680	9644	10608	11572	12538	13502	14468	15430
58	4064	5080	6096	7112	8128	9144	10160	11176	12192	13208	14224	15240	16256
59	4278	5348	6417	7488	8557	9626	10696	11766	12836	13905	14975	16045	17114
60	4500	5625	6750	7875	9000	10125	11250	12375	13500	14625	15750	16875	18000
61	4728	5910	7092	8274	9456	10638	11820	13002	14183	15368	16550	17730	18912
62	4968	6208	7448	8688	9930	11171	12412	13654	14895	16136	17376	18618	19860
63	5209	6511	7813	9115	10418	11720	13022	14325	15628	16929	18232	19534	20836
64	5464	6824	8192	9560	10920	12288	13656	15016	16384	17752	19112	20480	21848
65	5721	7151	8581	10012	11442	12872	14302	15732	17162	18594	20025	21455	22884
66	5990	7487	8984	9481	11979	13476	14973	16471	17969	19466	20963	22460	23958
67	6266	7832	9398	10965	12530	14098	15665	17230	18800	20366	21930	23496	25062
68	6552	8190	9824	11464	13104	14736	16376	18016	19656	21288	22928	24568	26200
69	6844	8555	10266	11977	13688	15399	17119	18821	20532	22243	23954	25665	27376

TABLE 11. MOMENTS OF INERTIA OF WEBS—(*Continued*)

Web depth ins.	Web thickness.												
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
70	7144	8930	10716	12502	14288	16074	17860	19646	21432	23218	25004	26790	28576
71	7455	9318	11180	13045	14910	16772	18636	20500	22365	24228	26090	2795o	29820
72	7776	9720	11664	13608	15552	17496	19440	21384	23328	25272	27216	29160	31104
73	8103	10130	12155	14180	16208	18232	20260	22285	24313	26338	28365	30390	32418
74	8442	10552	12662	14772	16884	18994	21105	23215	25326	27436	29547	31658	33769
75	8788	10985	13182	15380	17576	19772	21970	24168	26366	28562	30760	32956	35152
76	9144	11432	13720	16000	18288	20576	22864	25152	27440	29720	32008	34296	36581
77	9508	11885	14262	16640	19018	21394	23770	26150	28528	30910	33288	35666	38044
78	9887	12358	14829	17300	19771	22242	24714	27186	29658	32130	34602	37074	39546
79	10272	12840	15408	17976	20544	23112	25680	28248	30816	33384	35952	38520	41088
80	10664	13336	16000	18664	21336	24000	26664	29336	32000	34664	37336	40000	42664
81	11070	13838	16605	19375	22140	24908	27680	30445	33210	35980	38750	41515	44285
82	11487	14360	17232	20103	22975	25847	28720	31591	34464	37335	40207	43079	45951
83	11910	14890	17865	20845	23820	26800	29780	32755	35732	38714	41690	44670	47650
84	12352	15432	18520	21608	24696	27784	30872	33960	37048	40128	43216	46304	49392
85	12792	15990	19190	22386	25585	28782	31980	35180	38380	41580	44780	47979	51177
86	13251	16560	19880	23189	26502	29815	33128	36441	39760	43067	46380	49692	53006
87	13716	17145	20575	24002	27435	30862	34299	37720	41150	44580	48010	51440	54870
88	14200	17744	21296	24848	28398	31944	35496	39040	42592	46144	49692	53240	56792
89	14685	18356	22028	25700	29370	33040	36710	40385	44060	47730	51400	55070	58750
90	15188	18984	22780	26576	30372	34168	37964	41769	45556	49352	53162	56964	60750
91	15696	19620	23545	27470	31392	35330	39250	43170	47100	51020	54950	58875	62800
92	16224	22280	24336	28392	32448	36504	40560	44616	48672	52720	56776	60832	64888
93	16755	20945	25134	29324	33510	37700	41890	46080	50270	54460	58650	62840	67030
94	17304	21630	25952	30280	34608	38934	43260	47586	51912	56238	60560	64880	69216
95	17860	23325	26790	31260	35720	40185	44650	49120	53590	58050	62520	66980	71450
96	18432	23040	27648	32256	36864	41472	46080	50688	55296	59904	64512	69120	73728
97	19010	23762	28516	33270	38020	42775	47535	52288	57042	61790	66548	71300	76056
98	19608	24510	29412	34314	39216	44118	49020	53922	58824	63726	68628	73530	78432
99	20216	25270	30325	35380	40430	45485	50540	55595	60650	65705	70760	75815	80870
100	20832	26040	31248	36456	41664	46872	52080	57288	62496	67708	72912	78120	83336
101	21460	26829	32190	37558	42924	48290	53656	59020	64390	69758	75120	80485	85854
102	22108	27635	33162	38680	44216	49743	55270	60797	66324	71851	77378	82905	88432
103	22767	28460	34150	39840	45532	51225	56918	62608	68296	73988	79678	85369	91060
104	23432	29296	35152	41008	46872	52728	58584	64448	70304	76160	82024	87880	93736
105	24115	30144	36170	42200	48231	54260	60283	66320	72352	78375	84405	90435	96469
106	24813	31016	37219	43422	49625	55828	62031	68234	74437	80640	86843	93046	99249
107	25520	31902	38285	44668	51050	57436	63810	70185	76566	82946	89326	95707	102088
108	26248	32808	39368	45928	52488	59048	65608	72168	78736	85296	91856	98416	104976
109	26980	33720	40466	47220	53955	60700	67445	74192	80940	87680	94423	101170	107920
110	27729	34664	41600	48528	55452	62389	69321	76253	83185	90117	97049	103981	110913
111	28490	35610	42732	49856	56979	64107	71230	78348	85471	92594	99715	106845	113968
112	29269	36586	43903	51220	58537	65854	73171	80488	87805	95122	102439	109756	117073
113	30056	37570	45090	52600	60120	67635	75150	82660	90180	97695	105210	112720	120235
114	30864	38580	46296	54014	61730	69445	77160	84880	92590	100310	108020	115746	123460
115	31680	39600	47520	55441	63360	71290	79211	87126	95053	102975	110890	118846	126739
116	32512	40640	48768	56905	65034	73163	81292	89421	97550	105679	113808	121937	130166
117	33367	41709	50051	58393	66735	75077	83419	91761	100103	108445	116787	125129	133471
118	34230	42787	51344	59901	68458	77015	85572	94129	102686	111243	119800	128357	136914
119	35106	43884	52660	61438	70215	78988	87765	96545	105323	114100	122875	131650	140431
120	36000	45000	54000	63000	72000	81000	90000	99000	108000	117000	126000	135000	144000

TABLE 12

MOMENTS OF INERTIA 4—3″ x 3″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area. To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found in the table by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.)

Size of Rivet and number of rivet holes in each angle.	Percentage.
$\frac{3}{4}$″ rivet—1—$\frac{7}{8}$″ hole	84.3%
$\frac{3}{4}$″ rivet—2—$\frac{7}{8}$″ holes	68.4%
$\frac{7}{8}$″ rivet—1—1″ hole	82.0%

B. to B. Angles.	Thickness of Angles.							
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$
12	156	190	226	258	288	316	344	372
12½	172	210	248	284	318	350	380	412
13	186	228	270	310	346	382	416	450
13½	204	250	294	338	372	418	456	492
14	222	270	318	366	410	454	494	534
14½	240	294	348	398	426	492	534	580
15	260	316	372	428	480	532	576	626
15½	278	322	410	460	518	574	624	674
16	298	364	432	494	554	614	670	722
16½	318	390	464	532	594	660	718	774
17	342	416	494	568	634	704	768	828
17½	364	446	526	606	678	752	820	886
18	386	474	558	644	722	800	874	942
18½	410	504	594	684	768	850	930	1004
19	434	534	630	724	814	900	984	1066
19½	460	566	668	768	862	954	1040	1132
20	486	598	706	812	910	1008	1096	1196
20½	514	632	746	858	960	1066	1152	1260
21	542	664	784	904	1008	1124	1228	1324
21½	570	700	806	952	1062	1184	1294	1394
22	596	734	868	996	1116	1244	1360	1464
22½	626	772	912	1048	1174	1308	1428	1540
23	656	810	956	1098	1232	1372	1494	1618
23½	688	850	1002	1152	1292	1438	1570	1700

MOMENTS OF INERTIA 4—3″ x 3″ ANGLES 2 IN EACH FLANGE FOR VARIOUS DISTANCES BACK TO BACK—TABLE 12 (*Continued*)

B. to B. Angles.	Thickness of Angles.							
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$
24	720	888	1048	1204	1352	1504	1644	1782
24½	754	928	1098	1260	1416	1574	1720	1864
25	788	968	1146	1316	1480	1644	1794	1946
25½	820	1014	1196	1374	1546	1716	1874	2034
26	852	1054	1244	1432	1610	1786	1954	2120
26½	882	1098	1296	1492	1676	1862	2038	2208
27	922	1142	1348	1550	1744	1938	2120	2296
27½	962	1190	1406	1604	1813	2016	2202	2392
28	1000	1236	1462	1678	1884	2094	2284	2486
28½	1040	1262	1516	1750	1956	2164	2374	2582
29	1080	1328	1568	1804	2026	2251	2464	2676
29½	1120	1380	1630	1874	2106	2340	2560	2778
30	1160	1432	1692	1944	2184	2424	2654	2882

TABLE 13

MOMENTS OF INERTIA FOR 4—3½″ x 3½″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area: To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found in the tables by the percentage corresponding to the size and number of rivet holes in each angle. Rivet holes are computed as ⅛″ larger than the diameter of the rivet.

Size of Rivet and number of rivet holes in each angle.	Percentage.
¾″ rivet—1—⅞″ hole	86.4%
¾″ rivet—2—⅞″ holes	72.8%
⅞″ rivet—1—1″ hole	84.4%
⅞″ rivet—2—1″ holes	68.8%

B. to B. Angles.	Thickness of Angles.									
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8
12	221	259	297	332	367	400	432	461	491	517
12½	245	285	327	366	404	441	476	509	542	571
13	267	312	357	400	442	482	520	557	592	626
13½	287	340	390	436	483	527	569	609	648	685
14	311	368	422	473	524	572	619	661	704	744
14½	339	400	457	513	569	621	670	718	765	808
15	365	431	493	553	614	670	721	776	826	873
15½	393	464	532	596	662	724	780	838	893	943
16	421	497	571	640	710	777	839	900	960	1014
16½	451	534	612	687	762	833	902	965	1030	1088
17	481	571	654	735	814	890	964	1030	1100	1162
17½	513	611	698	785	870	951	1029	1102	1175	1244
18	545	651	743	835	926	1012	1094	1175	1251	1327
18½	585	692	790	887	983	1077	1166	1250	1331	1412
19	625	732	838	940	1041	1142	1238	1325	1411	1497
19½	657	777	889	995	1104	1210	1312	1406	1498	1590
20	687	823	940	1050	1166	1278	1385	1487	1586	1682
20½	727	868	992	1110	1233	1353	1462	1571	1676	1779
21	765	913	1043	1170	1300	1427	1539	1655	1766	1876
21½	805	960	1098	1232	1370	1502	1624	1745	1863	1978
22	845	1007	1153	1295	1440	1577	1709	1835	1961	2080
22½	891	1058	1213	1362	1518	1660	1794	1930	2061	2188
23	935	1109	1273	1430	1596	1742	1879	2025	2161	2297
23½	979	1161	1333	1500	1675	1829	1969	2122	2266	2409
24	1021	1212	1393	1570	1746	1917	2059	2220	2371	2522
24½	1071	1269	1458	1642	1826	2005	2159	2327	2483	2641
25	1119	1322	1523	1715	1906	2092	2259	2435	2595	2761
25½	1169	1377	1589	1790	1991	2184	2359	2538	2713	2882
26	1217	1442	1655	1865	2076	2276	2459	2640	2831	3002
26½	1267	1502	1726	1945	2166	2371	2564	2752	2951	3137

MOMENTS OF INERTIA FOR 4—3½″ x 3½″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK—TABLE 13 (*Continued*)

B. to B. Angles.	Thickness of Angles.									
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8
27	1319	1562	1798	2025	2256	2467	2669	2865	3071	3271
27½	1373	1626	1872	2108	2346	2567	2774	2983	3196	3402
28	1425	1689	1946	2190	2436	2667	2879	3100	3321	3532
28½	1481	1756	2021	2278	2531	2772	2999	3225	3448	3677
29	1535	1822	2096	2365	2626	2877	3119	3350	3576	3822
29½	1595	1889	2174	2450	2726	2987	3231	3475	3713	3962
30	1649	1957	2253	2535	2826	3097	3344	3600	3850	4102
30½	1709	2025	2333	2627	2926	3208	3462	3732	4001	4252
31	1769	2092	2413	2720	3026	3319	3579	3865	4151	4402
31½	1833	2167	2498	2817	3134	3438	3711	4002	4291	4561
32	1895	2242	2583	2915	3241	3557	3844	4140	4431	4721
32½	1963	2327	2673	3015	3353	3675	3972	4280	4581	4891
33	2031	2392	2763	3115	3466	3792	4099	4420	4731	5061
33½	2095	2472	2851	3215	3578	3918	4234	4570	4886	5216
34	2159	2552	2938	3315	3691	4045	4369	4720	5041	5372
34½	2225	2632	3033	3420	3803	4171	4514	4870	5156	5547
35	2287	2712	3128	3525	3916	4298	4659	5020	5371	5722
35½	2359	2792	3221	3635	4045	4433	4802	5170	5486	5897
36	2429	2872	3313	3745	4166	4567	4944	5320	5701	6072
36½	2505	2962	3413	3855	4295	4704	5087	5435	5871	6252
37	2579	3052	3513	3965	4416	4842	5229	5650	6041	6432
37½	2649	3140	3613	4080	4541	4980	5381	5765	6221	6631
38	2719	3229	3713	4195	4666	5117	5534	5980	6401	6820
38½	2795	3316	3818	4313	4801	5267	5696	6150	6586	7016
39	2869	3403	3923	4430	4936	5417	5859	6320	6771	7212
39½	2949	3496	4030	4548	5064	5567	6016	6435	6946	7507
40	3029	3589	4138	4665	5191	5717	6174	6650	7121	7602
40½	3119	3680	4250	4790	5334	5867	6341	6835	7321	7802
41	3189	3772	4363	4915	5476	6017	6509	7020	7521	8002
41½	3271	3872	4475	5040	5621	6119	6679	7205	7761	8212
42	3355	3972	4588	5165	5766	6320	6849	7390	7901	8422

TABLE 14

MOMENTS OF INERTIA 4—4″ x 4″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area. To obtain moments of inertia of net areas for various sizes and number of rivet holes, multiply the moment of inertia of the gross area as found in the table by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.)

Size of Rivet and number of rivet holes in each angle.	Percentage.
$\frac{3}{4}$″ rivet—1—$\frac{7}{8}$″ hole	88.2%
$\frac{3}{4}$″ rivet—2—$\frac{7}{8}$″ holes	76.5%
$\frac{7}{8}$″ rivet—1—1″ hole	86.5%
7″ rivet—2—1″ holes	73.0%
1″ rivet—1—1$\frac{1}{8}$″ hole	84.9%
1″ rivet—2—1$\frac{1}{8}$″ holes	69.9%

B. to B. Angles	Thickness of Angles.									
	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$
12	243	287	330	371	410	448	483	519	551	583
12$\frac{1}{2}$	268	316	363	408	450	493	532	571	608	643
13	293	345	398	447	493	540	584	626	667	707
13$\frac{1}{2}$	321	377	434	488	539	590	638	685	729	773
14	350	409	472	530	587	642	694	745	795	842
14$\frac{1}{2}$	378	444	512	575	637	697	754	809	863	915
15	406	479	552	621	687	754	815	876	934	990
15$\frac{1}{2}$	438	517	595	670	741	813	879	945	1008	1069
16	470	555	640	720	797	874	946	1017	1085	1151
16$\frac{1}{2}$	504	596	686	772	856	938	1015	1091	1165	1236
17	538	637	733	826	916	1004	1087	1169	1247	1324
17$\frac{1}{2}$	574	689	783	882	977	1072	1161	1249	1333	1416
18	611	724	834	939	1042	1143	1238	1331	1422	1510
18$\frac{1}{2}$	656	770	887	999	1109	1216	1317	1417	1513	1607
19	690	817	941	1060	1177	1291	1398	1505	1607	1708
19$\frac{1}{2}$	732	865	997	1124	1247	1369	1483	1596	1705	1812
20	773	915	1055	1189	1319	1449	1570	1690	1805	1919
20$\frac{1}{2}$	816	968	1114	1256	1393	1531	1659	1786	1909	2029

MOMENTS OF INERTIA 4—4″ x 4″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK—TABLE 14 (*Continued*)

B. to B. Angles	Thickness of Angles.									
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8
21	860	1020	1175	1325	1470	1615	1751	1885	2015	2142
21½	907	1073	1238	1396	1549	1702	1845	1987	2124	2256
22	955	1130	1302	1469	1630	1791	1942	2091	2236	2378
22½	1002	1184	1368	1543	1714	1883	2041	2198	2350	2500
23	1050	1245	1436	1620	1799	1977	2143	2309	2468	2626
23½	1100	1305	1505	1698	1887	2072	2247	2421	2584	2755
24	1150	1366	1576	1778	1976	2171	2354	2536	2712	2886
24½	1205	1429	1647	1861	2068	2271	2463	2654	2839	3022
25	1260	1494	1723	1945	2160	2375	2576	2775	2969	3160
25½	1316	1559	1799	2031	2257	2480	2690	2899	3101	3301
26	1373	1626	1876	2118	2355	2587	2807	3025	3236	3445
26½	1429	1695	1956	2208	2454	2697	2926	3154	3374	3592
27	1486	1765	2037	2299	2556	2809	3049	3286	3516	3743
27½	1548	1836	2120	2393	2661	2924	3173	3420	3660	3897
28	1610	1908	2204	2488	2767	3041	3300	3558	3807	4054
28½	1672	1984	2289	2585	2875	3160	3429	3697	3957	4214
29	1735	2057	2376	2683	2986	3282	3561	3840	4110	4377
29½	1800	2137	2466	2785	3098	3406	3696	3985	4265	4542
30	1865	2215	2556	2887	3212	3531	3833	4133	4424	4712
30½	1935	2294	2649	2992	3329	3660	3972	4284	4586	4885
31	2005	2376	2743	3098	3448	3791	4114	4438	4750	5060
31½	2073	2459	2839	3207	3568	3923	4260	4594	4918	5239
32	2140	2544	2936	3317	3691	4059	4406	4753	5088	5421
32½	2215	2626	3035	3429	3817	4196	4556	4915	5261	5606
33	2290	2717	3136	3543	3943	4337	4608	5079	5437	5793
33½	2365	2802	3239	3658	4072	4477	4863	5246	5616	5985
34	2440	2892	3342	3777	4204	4623	5020	5416	5799	6179
34½	2515	2986	3448	3897	4338	4770	5179	5588	5983	6376
35	2590	3077	3555	4018	4473	4919	5343	5763	6171	6576
35½	2665	3170	3664	4141	4611	5070	5507	5941	6363	6780
36	2765	3269	3776	4266	4749	5225	5674	6122	6557	6987
36½	2843	3367	3888	4393	4892	5381	5843	6304	6753	7197
37½	3005	3564	4118	4654	5184	5700	6191	6681	7155	7627
38½	3177	3769	4354	4921	5477	6027	6547	7066	7567	8066
39½	3352	3978	4597	5193	5785	6365	6915	7463	7994	8520
40½	3530	4195	4846	5478	6102	6713	7291	7870	8430	8989
41½	3715	4417	5102	5768	6425	7068	7681	8289	8880	9466
42½	3907	4643	5364	6065	6757	7434	8077	8719	9339	9958
43½	4100	4874	5636	6370	7097	7809	8483	9157	9812	10462
44½	4310	5111	5910	6683	7445	8193	8901	9609	10296	10979
45½	4510	5359	6193	7001	7800	8587	9329	10071	10791	11506
46½	4718	5608	6482	7328	8167	8987	9767	10543	11300	12051
47½	4943	5865	6778	7665	8539	9399	10213	11026	11816	12603
48½	5165	6127	7081	8006	8922	9810	10672	11521	12347	13169
49½	5515	6395	7389	8356	9313	10250	11139	12026	12888	13747
50½	5630	6667	7706	8714	9711	10689	11617	12542	13444	14338
51½	5840	6945	8026	9078	10118	11137	12105	13069	14010	14944
52½	6078	7229	8356	9450	10534	11595	12603	13609	14588	15558

TABLE 14 (*Continued*)

MOMENTS OF INERTIA 4—4″ x 4″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B. to B. Angles	Thickness of Angles.									
	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$
53½	6328	7521	8691	9830	10957	12068	13112	14159	15173	16187
54½	6578	7817	9035	10217	11390	12537	13629	14718	15776	16829
55½	6828	8119	9382	10612	11830	13025	14158	15287	16388	17482
56½	7090	8427	9737	11014	12279	13529	14695	15871	17013	18150
57½	7353	8730	10098	11422	12735	14024	15243	16461	17647	18828
58½	7640	9059	10469	11842	13203	14537	15801	17067	18293	19518
59½	7928	9384	10844	12267	13680	15058	16370	17679	18953	20221
60½	8187	9713	11223	12700	14160	15592	16949	18306	19624	20938
61½	8462	10047	11615	13137	14650	16132	17539	18941	20308	21667
62½	8747	10391	12008	13584	15139	16678	18134	19589	20998	22409
63½	9035	10737	12410	14040	15660	17242	18745	20245	21708	23160
64½	9328	11088	12820	14502	16173	17809	19361	20915	22425	23926
65½	9640	11448	13235	14974	16697	18387	19994	21596	23154	24706
66½	9958	11814	13657	15452	17233	18977	20629	22286	23895	25499
67½	10252	12184	14084	15937	17769	19567	21279	22991	24644	26302
68½	10565	12557	14520	16427	18325	20177	21939	23701	25412	27119
69½	10890	12945	14963	16927	18885	20793	22610	24421	26187	27949
70½	11228	13327	15410	17438	19450	21417	23289	25161	26974	28789
71½	11553	13719	15862	17951	20009	22052	23979	25902	27773	29639
72½	11880	14115	16316	18466	20570	22690	24670	26645	28575	30490

TABLE 15

MOMENTS OF INERTIA FOR 4—6″ x 6″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area. To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found in the table by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.)

Size of Rivet and Number of Rivet Holes in each Angle.	Percentage.
$\frac{3}{4}$″ rivet—1—$\frac{7}{8}$″ hole	92.2%
$\frac{3}{4}$″ rivet—2—$\frac{7}{8}$″ holes	84.4%
$\frac{7}{8}$″ rivet—1—1″ hole	91.2%
$\frac{7}{8}$″ rivet—2—1″ holes	82.2%
1″ rivet—1—$1\frac{1}{8}$″ hole	90.6%
1″ rivet—2—$1\frac{1}{8}$″ holes	80.6%

B.B. Ang.	Thickness of Angles.										
	$\frac{3}{8}$″	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″	$\frac{7}{8}$″	$\frac{15}{16}$″	1″
16	768	885	999	1107	1215	1321	1419	1517	1616	1710	1801
$\frac{1}{2}$	824	950	1073	1190	1306	1420	1526	1632	1739	1841	1938
17	883	1018	1150	1276	1400	1523	1638	1752	1861	1976	2082
$\frac{1}{2}$	944	1088	1230	1365	1498	1630	1753	1876	1999	2117	2231
18	1007	1162	1312	1457	1600	1741	1873	2005	2136	2263	2385
$\frac{1}{2}$	1072	1137	1398	1552	1705	1856	1997	2138	2279	2414	2545
19	1139	1315	1486	1651	1814	1974	2125	2276	2426	2571	2710
$\frac{1}{2}$	1209	1396	1578	1753	1926	2097	2257	2418	2578	2732	2881
20	1281	1479	1672	1858	2042	2223	2394	2565	2735	2899	3057
$\frac{1}{2}$	1355	1565	1769	1967	2161	2353	2535	2716	2896	3071	3239
21	1431	1653	1869	2078	2285	2488	2680	2872	3063	3249	3427
$\frac{1}{2}$	1509	1743	1972	2193	2411	2626	2829	3032	3235	3431	3619
22	1590	1837	2078	2311	2541	2768	2983	3197	3411	3618	3818
$\frac{1}{2}$	1673	1933	2187	2432	2674	2913	3141	3366	3592	3811	4022
23	1758	2031	2298	2557	2812	3063	3303	3541	3779	4010	4231
$\frac{1}{2}$	1845	2131	2412	2684	2952	3217	3469	3720	3969	4212	4446
24	1934	2235	2530	2816	3097	3374	3639	3903	4165	4421	4665
$\frac{1}{2}$	2025	2341	2650	2950	3244	3536	3814	4090	4366	4634	4892
25	2119	2449	2772	3087	3396	3701	3992	4283	4571	4853	5123
$\frac{1}{2}$	2215	2560	2897	3228	3551	3870	4178	4479	4782	5077	5360
26	2313	2674	3027	3371	3709	4044	4363	4681	4998	5306	5602
$\frac{1}{2}$	2413	2790	3159	3512	3871	4221	4554	4887	5218	5540	5850
27	2516	2909	3296	3668	4037	4402	4750	5097	5443	5780	6103
$\frac{1}{2}$	2619	3029	3431	3822	4206	4587	4950	5312	5673	6024	6363
28	2727	3153	3572	3979	4379	4775	5154	5532	5908	6275	6626
$\frac{1}{2}$	2835	3279	3714	4139	4555	4968	5362	5755	6147	6529	6897
29	2946	3408	3860	4302	4735	5164	5575	5984	6391	6789	7171
$\frac{1}{2}$	3060	3539	4009	4468	4918	5364	5792	6217	6642	7055	7452
30	3176	3673	4161	4638	5105	5569	6013	6455	6896	7326	7739
$\frac{1}{2}$	3293	3809	4315	4811	5296	5777	6238	6698	7154	7601	8030

TABLE 15 (*Continued*)

MOMENTS OF INERTIA FOR 4—6″ x 6″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B.B Ang	Thickness of Angles.										
	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
31	3412	3948	4472	4986	5489	5989	6469	6944	7419	7882	8328
1/2	3534	4089	4633	5166	5687	6205	6701	7195	7686	8168	8630
32	3658	4233	4796	5348	5888	6424	6939	7450	7961	8459	8939
1/2	3785	4379	4962	5534	5093	6647	7181	7712	8240	8757	9252
33	3913	4529	5132	5723	6301	6875	7427	7976	8523	9057	9572
1/2	4044	4680	5304	5914	6513	7107	7679	8246	8812	9365	9896
34	4177	4834	5478	6110	6729	7342	7933	8520	9105	9677	10227
1/2	4312	4990	5656	6308	6948	7579	8193	8799	9402	9995	10563
35	4448	5150	5836	6510	7170	7824	8455	9082	9706	10316	10904
1/2	4588	5311	6020	6715	7395	8071	8723	9369	10014	10644	11252
36	4731	5475	6206	6924	7625	8318	8993	9661	10327	10977	11602
1/2	4874	5642	6395	7135	7858	8577	9270	9958	10643	11315	11960
37 1/2	5167	5982	6782	7567	8335	9098	9835	10564	11293	12006	12692
38 1/2	5470	6333	7180	8012	8826	9635	10415	11191	11963	12720	13447
39 1/2	5782	6694	7590	8471	9331	10187	11013	11834	12651	13451	14222
40 1/2	6102	7066	8012	8941	9851	10755	11628	12495	13359	14206	15020
41 1/2	6431	7446	8444	9426	10384	11338	12261	13175	14088	14980	15842
42 1/2	6770	7838	8888	9922	10933	11938	12909	13874	14833	15777	16682
43 1/2	7116	8239	9344	10433	11496	12553	13575	14590	15601	16593	17547
44 1/2	7471	8651	9812	10956	12072	13183	14255	15325	16388	17429	18432
45 1/2	7834	9073	10292	11491	12662	13828	14958	16076	17193	18285	19342
46 1/2	8206	9505	10779	12038	13267	14489	15673	16847	18018	19165	20272
47 1/2	8588	9947	11281	12599	13886	15165	16408	17635	18863	20065	21224
48 1/2	8978	10399	11795	13173	14517	15857	17156	18444	19728	20985	22197
49 1/2	9377	10861	12320	13760	15168	16565	17924	19270	20613	21925	23192
50 1/2	9784	11334	12856	14360	15827	17289	18706	20114	21514	22886	24212
51 1/2	10200	11816	13402	14973	16504	18030	19508	20975	22434	23870	25252
52 1/2	10625	12309	13962	15600	17194	18783	20327	21855	23380	24874	26313
53 1/2	11060	12811	14534	16238	17897	19553	21161	22752	24338	25895	27397
54 1/2	11500	13323	15118	16888	18617	20339	22013	23667	25321	26940	28503
55 1/2	11951	13849	15710	17553	19350	21132	22878	24603	26321	28005	29632
56 1/2	12412	14381	16318	18233	20097	21957	23764	25555	27343	29090	30782
57 1/2	12879	14923	16933	18921	20857	22790	24668	26525	28379	30200	31957
58 1/2	13356	15477	17561	19624	21636	23638	25586	27516	29443	31330	33152
59 1/2	13842	16041	18200	20339	22423	24501	26523	28520	30518	32475	34367
60 1/2	14339	16612	18852	21068	23227	25380	27475	29545	31613	33645	35602
61 1/2	14842	17200	19518	21810	24048	26271	28443	30590	32736	34835	36870
62 1/2	15352	17793	20190	22564	24878	27186	29431	31652	33870	36045	38147
63 1/2	15872	18396	20876	23335	25727	28113	30433	32730	35028	37279	39452
64 1/2	16403	19011	21575	24113	26587	29053	31453	33830	36200	38530	40777
65 1/2	16942	19633	22280	24903	27459	30010	32490	34948	37398	39798	42122
66 1/2	17490	20271	23002	25711	28352	30981	33543	36080	38613	41095	43492

TABLE 15 (*Continued*)

MOMENTS OF INERTIA FOR 4—6″ x 6″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B.B. Ang.	Thickness of Angles.										
	3″/8	7″/16	1″/2	9″/16	5″/8	11″/16	3″/4	13″/16	7″/8	15″/16	1″
67½	18043	20911	23735	26529	29257	31973	34616	37230	39848	42411	44892
68½	18610	21567	24477	27361	30172	32975	35703	38405	41100	43749	46306
69½	19182	22235	25235	28209	31107	33996	36810	39595	42378	45107	47742
70½	19763	22909	26001	29068	32056	35031	37928	40800	43668	46480	49197
71½	20355	23597	26780	29938	33016	36083	39073	42030	44983	47880	50682
72½	20950	24286	27565	30820	33987	37140	40224	43270	46313	49295	52172
73½	21562	24992	28370	31718	34977	38229	41398	44530	47668	50735	53702
74½	22177	25711	29180	32628	35979	39325	42585	45810	49033	52195	55252
75½	22804	26436	30005	33548	36997	40440	43793	47110	50428	53680	56822
76½	23433	27166	30840	34483	38027	41567	45013	48430	51833	55175	58412
77½	24080	27916	31690	35429	39077	42709	46253	49760	53268	56705	60022
78½	24732	28671	32543	36393	40133	43870	47513	51110	54713	58245	61652
79½	25397	29436	33420	37364	41207	45043	48788	52485	56183	59810	63312
80½	26062	30219	34300	38352	42297	46235	50077	53880	57673	61395	64992
81½	26741	30999	35190	39351	43390	47442	51383	55280	59173	63000	66692
82½	27427	31796	36095	40363	44506	48665	52703	56705	60698	64625	68412
83½	28123	32606	37010	41388	45638	49895	54048	58150	62248	66270	70162
84½	28827	33419	37940	42426	46788	51150	55403	59620	63813	67940	71922
85½	29536	34243	38875	43473	47942	52415	56778	61090	65388	69625	73712
86½	30261	35086	39830	44538	49118	53705	58173	62590	67008	71335	75522
87½	30992	35926	40790	45620	50303	55005	59578	64105	68628	73065	77352
88½	31732	36785	41765	46708	51508	56315	61013	65650	70268	74815	79202
89½	32479	37653	42748	47808	52718	57647	62443	67190	71928	76585	81082
90½	33232	38531	43745	48919	53953	58985	63903	68765	73608	78385	82972
91½	33997	39416	44750	50053	55198	60355	65383	70350	75318	80195	84902
92½	34772	40317	45770	51193	56458	61735	66873	71960	77028	82020	86842
93½	35552	41219	46805	52348	57733	63125	68383	73580	78778	83875	88802
94½	36347	42141	47846	53513	59018	64530	69903	75225	80538	85745	90782
95½	37142	43071	48897	54688	60318	65955	71453	76890	82318	87645	92792
96½	37952	44001	49967	55888	61638	67395	73013	78570	84108	89565	94832
97½	38767	44954	51040	57088	62963	68855	74593	80260	85928	91495	96882
98½	39596	45911	52125	58308	64308	70320	76193	81980	87768	93455	98962
99½	40433	46886	53230	59538	65668	71810	77798	83710	89628	95435	101042
100½	41276	47859	54340	60788	67043	73315	79413	85470	91498	97435	103172
101½	42126	48846	55465	62038	68428	74825	81073	87235	93398	99455	105322
102½	42988	49846	56600	63313	69843	76355	82733	89020	95318	101495	107462
103½	43855	50851	57740	64593	71248	77910	84413	90835	97258	103555	109652
104½	44736	51871	58900	65888	72658	79475	86113	92660	99208	105635	111852
105½	45626	52901	60070	67193	74103	81045	87813	94500	101178	107745	114072
106½	46518	53941	61250	68518	75568	82640	89543	96370	103178	109855	116332
107½	47426	54986	62450	69848	77048	84255	91293	98250	105188	112015	118622
108½	48340	56050	63640	71210	78528	85875	93055	100130	107230	114155	120882

TABLE 16

MOMENTS OF INERTIA FOR 4 – 8″ x 8″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area. To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found in the tables by the percentage corresponding to the size and number of rivet holes in each angle. Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.

Size of Rivet and Number of Rivet Holes in each Angle.	Percentage.
$\frac{7}{8}$″ rivet—1—1″ hole	93.5%
$\frac{7}{8}$″ rivet—2—1″ hole	86.8%
$\frac{7}{8}$″ rivet—3—1″ hole	80.2%
1″ rivet—1—1$\frac{1}{8}$″ hole	92.5%
1″ rivet—2—1$\frac{1}{8}$″ hole	85.2%
1″ rivet—3—1$\frac{1}{8}$″ hole	77.9%
1$\frac{1}{8}$″ rivet—2—1$\frac{1}{4}$″ hole	83.5%

B.B. Angles	Thickness of Angles.										
	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″	$\frac{7}{8}$″	$\frac{15}{16}$″	1″		1$\frac{1}{4}$″
18	1633	1819	2000	2178	2345	2516	2681	2844	2993		3300
18$\frac{1}{2}$	1740	1939	2132	2323	2503	2691	2862	3036	3196		3525
19	1851	2063	2269	2473	2664	2859	3048	3235	3406		
19$\frac{1}{2}$	1967	2192	2412	2628	2832	3039	3241	3440	3624		
20	2086	2325	2559	2789	3006	3227	3442	3653	3849		
20$\frac{1}{2}$	2209	2463	2710	2955	3185	3419	3648	3873	4081		
21	2335	2605	2867	3126	3371	3619	3862	4100	4322		
21$\frac{1}{2}$	2466	2751	3028	3302	3562	3824	4082	4334	4569		
22	2601	2901	3194	3484	3758	4036	4308	4576	4825		
22$\frac{1}{2}$	2740	3057	3365	3671	3961	4253	4542	4823	5087		
23	2882	3216	3541	3863	4169	4475	4781	5089	5378		
23$\frac{1}{2}$	3028	3380	3722	4060	4383	4707	5027	5341			
24	3178	3548	3907	4263	4603	4944	5281	5611	5920		
24$\frac{1}{2}$	3332	3720	4097	4471	4827	5187	5540	5887	6212		
25	3491	3897	4292	4685	5058	5436	5807	6171	6512		
25$\frac{1}{2}$	3652	4077	4492	4902	5295	5680	6079	6462	6820		
26	3818	4263	4696	5127	5537	5952	6359	6769	7136		
26$\frac{1}{2}$	3987	4452	4906	5355	5785	6218	6634	7064	7458		
27	4160	4647	5120	5590	6039	6491	6938	7376	7789		
27$\frac{1}{2}$	4338	4845	5339	5830	6299	6772	7237	7695	8026		
28	4519	5048	5563	6074	6564	7057	7543	8021	8372		
28$\frac{1}{2}$	4704	5255	5791	6324	6835	7348	7855	8354	8824		9779
29	4893	5466	6025	6580	7113	7646	8174	8694	9198		
29$\frac{1}{2}$	5086	5682	6263	6840	7395	7951	8500	9041	9551		10587
30	5282	5902	6506	7107	7682	8261	8832	9395	9927		
30$\frac{1}{2}$	5482	6118	6754	7377	7977	8576	9172	9757	10309		
31	5687	6356	7007	7654	8276	8909	9518	10125	10699		
31$\frac{1}{2}$	5896	6588	7264	7935	8581	9229	9870	10500	11096		
32	6107	6826	7526	8221	8892	9564	10228	10882	11501		
32$\frac{1}{2}$	6323	7068	7793	8514	9209	9905	10593	11272	11914		
33	6543	7314	8065	8812	9531	10259	10966	11668	12336	13019	13684
33$\frac{1}{2}$	6766	7554	8342	9114	9859	10605	11345	12072	12761	13470	14163
34	6994	7819	8623	9422	10193	10962	11729	12482	13196	13932	14643
34$\frac{1}{2}$	7226	8078	8910	9735	10534	11330	12123	12899	13641	14399	15137
35	7461	8342	9200	10054	10878	11702	12520	13325	14089	14874	15637
35$\frac{1}{2}$	7700	8610	9496	10377	11229	12080	12927	13758	14547	15359	16147

TABLE 16 (*Continued*)

MOMENTS OF INERTIA FOR 4—8″ x 8″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B.B. Angles	Thickness of Angles.										
	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″	1 1/16″	1 1/8″
36	7944	8882	9797	10706	11585	12465	13337	14197	15012	15851	16667
36 1/2	8190	9158	10102	11040	11949	12855	13756	14642	15486	16352	17192
37 1/2	8696	9725	10728	11725	12690	13655	14613	15556	16452	17374	18267
38 1/2	9212	10308	11370	12430	13454	14479	15497	16497	17451	18429	19382
39 1/2	9754	10909	12036	13158	14244	15329	16407	17467	18478	19515	20526
40 1/2	10305	11527	12719	13904	15054	16201	17343	18465	19536	20634	21702
41 1/2	10871	12164	13420	14671	15888	17100	18305	19492	20626	21783	22913
42 1/2	11456	12816	14141	15463	16744	18021	19293	20547	21741	22966	24157
43 1/2	12055	13486	14883	16272	17624	18971	20312	21629	22889	24179	25436
44 1/2	12667	14175	15643	17107	18527	19944	21351	22742	24067	25425	26747
45 1/2	13298	14881	16420	17959	19450	20940	22418	23878	25276	26703	28092
46 1/2	13943	15603	17220	18831	20399	21962	23518	25047	26514	28013	29472
47 1/2	14603	16343	18038	19728	21369	23008	24636	26242	27780	29354	30882
48 1/2	15280	17099	18876	20643	22364	24080	25786	27462	29078	30724	32328
49 1/2	15970	17874	19729	21580	23379	25172	26960	28719	30406	32132	33812
50 1/2	16680	18666	20603	22539	24419	26295	28161	29998	31761	33565	35321
51 1/2	17401	19476	21499	23519	25481	27440	29388	31307	33156	35034	36866
52 1/2	18140	20301	22413	24517	26576	28610	30643	32647	34566	36534	38443
53 1/2	18892	21146	23347	25539	27678	29805	31926	34012	36016	38064	40057
54 1/2	19661	22008	24298	26581	28807	31022	33228	35403	37491	39624	41704
55 1/2	20445	22887	25269	27645	29960	32270	34566	36827	39001	41224	43387
56 1/2	21245	23784	26260	28729	31139	33535	35923	38277	40538	42849	45082
57 1/2	22060	24698	27271	29834	32339	34830	37313	39754	42111	44508	46847
58 1/2	22895	25628	28298	30964	33561	36148	38723	41266	43703	46199	48626
59 1/2	23740	26577	29348	32109	34809	37490	40166	42797	45335	47918	50442
60 1/2	24600	27546	30417	33279	36074	38857	41628	44362	46992	49669	52292
61 1/2	25478	28526	31500	34473	37364	40250	43118	45951	48680	51464	54167
62 1/2	26375	29530	32608	35679	38679	41665	44643	47575	50396	53279	56082
63 1/2	27280	30546	33733	36910	40016	43106	46183	49217	52146	55124	58032
64 1/2	28205	31582	34878	38165	41379	44570	47760	50897	53921	57004	60012
65 1/2	29145	32636	36038	39439	42759	46061	49358	52602	55726	58914	62027
66 1/2	30100	33706	37221	40733	44164	47579	50979	54332	57566	60864	64072
67 1/2	31070	34795	38427	42049	45591	49122	52633	56097	59436	62844	66152
68 1/2	32055	35897	39646	43393	47049	50690	54318	57887	61336	64854	68272
69 1/2	33063	37021	40886	44749	48519	52275	56018	59697	63256	66894	70422
70 1/2	34080	38161	42148	46129	50019	53890	57748	61557	65226	68964	72612
71 1/2	35115	39321	43428	47529	51539	55530	59503	63427	67206	71064	74852
72 1/2	36157	40495	44723	48954	53079	57190	61288	65327	69236	73204	77082
73 1/2	37220	41688	46044	50394	54649	58880	63103	67257	71276	75364	79332
74 1/2	38300	42896	47378	51859	56229	60595	64938	69217	73356	77574	81672
75 1/2	39395	44126	48738	53344	57849	62330	66798	71202	75456	79794	84012
76 1/2	40505	45366	50108	54849	59479	64095	68698	73217	77606	82064	86402
77 1/2	41630	46631	51503	56379	61139	65880	70608	75257	79766	84354	88822
78 1/2	42775	47910	52918	57924	62824	67700	72548	77337	81966	86674	91282
79 1/2	43930	49206	54348	59480	64524	69530	74518	79427	84191	89034	93752
80 1/2	45105	50516	55803	61089	66249	71395	76518	81567	86456	91424	96272
81 1/2	46285	51848	57268	62699	68004	73280	78538	83717	88744	93844	98822
82 1/2	47495	53196	58768	64329	69779	75190	80578	85907	91056	96299	101402
83 1/2	48715	54566	60273	65979	71559	77130	82658	88117	93401	98784	104012
84 1/2	49945	55946	61798	67649	73379	79080	84758	90367	95786	101294	106672
85 1/2	51195	57346	63348	69349	75219	81065	86888	92637	98186	103854	109352
86 1/2	52455	58766	64918	71059	77089	83070	89048	94927	100636	106414	112082
87 1/2	53740	60196	66498	72804	78969	85100	91218	97257	103096	109024	114812
88 1/2	55040	61656	68108	74559	80879	87165	93418	99607	105586	111674	117602

TABLE 16 (*Continued*)

MOMENTS OF INERTIA FOR 4—8″ x 8″ ANGLES (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B.B. Angles	Thickness of Angles.										
	½″	9/16″	⅝″	11/16″	¾″	13/16″	⅞″	15/16″	1″	1 1/16″	1⅛″
89½	56645	63116	69738	76559	82809	89250	95668	101987	108056	114354	120432
90	57675	64606	71378	78139	84759	91350	97928	104387	110666	117054	123282
90½	59020	66106	73058	79959	86749	93480	100208	106887	113256	119784	126172
91	60375	67636	74718	81789	88749	95640	102518	109417	115866	122554	129082
91½	61755	69176	76423	83659	90769	97820	104858	111797	118526	125354	132032
92	63135	70736	78138	85549	92809	100030	107218	114337	121206	128194	135022
92½	64545	72306	79883	87459	94879	102250	109608	116867	123906	131064	138042
93	65965	73896	81638	89379	96979	104510	112038	119447	126656	133964	141092
93½	67405	75506	83428	91329	99089	106790	114478	122057	129406	136884	144182
94	68857	77126	85218	93289	101229	109100	116948	124697	132206	139844	147292
94½	70315	78766	87038	95299	103389	111429	119468	127367	135056	142844	150442
95	71775	80436	88868	97304	105569	113780	121968	130057	137906	145864	153642
95½	73245	82116	90728	99339	107779	116160	124528	132787	140796	148914	156862
96	74815	83811	92608	101389	110009	118570	127108	135537	143706	152014	160102
96½	76435	85526	94498	103469	112259	121000	129718	138307	146656	155124	163392
97	77885	87256	96418	105559	114529	123440	132338	141127	149636	158274	166712
97½	79455	89006	98348	107669	116829	125920	135008	143947	152656	161474	170092
98	81015	90766	100288	109809	119139	128420	137688	146827	155666	164674	173462
98½	82605	92556	102258	111969	121489	130950	140408	149707	158756	167914	176882
99	84215	94356	104248	114149	123859	133510	143128	152637	161846	171214	180332
99½	85835	96166	106258	116349	126258	136080	145888	155577	164956	174514	183812
100	87475	97996	108288	118569	128669	138680	148678	158547	168126	177844	187342
100½	89125	99856	110338	120819	131089	141300	151518	161557	171306	181224	190912
101	90785	101716	112388	123079	133559	143950	154328	164587	174536	184624	194192
101½	92470	103606	114488	125359	136029	146620	157208	167647	177766	188074	198092
102	94165	105516	116598	127659	138539	149320	160118	170737	181056	191534	201772
102½	95885	107426	118718	129979	141069	152050	163028	173847	184356	195034	205452
103	97625	109376	120848	132349	143609	154800	165968	176987	187686	198564	209182
103½	99365	111336	123018	134709	146189	157580	168968	180187	191076	202144	212942
104	101115	113306	125208	137099	148779	160390	171968	183387	194476	205734	216732
104½	102895	115296	127408	139509	151389	163200	174968	186617	197886	209364	220542
105	104700	117296	129608	141950	154050	166040	178068	189860	201346	213004	224362

TABLE 17

MOMENTS OF INERTIA FOR 4—5″ x 3½″ ANGLES LONG LEG AGAINST WEB, 2 IN EACH FLANGE FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross areas. To obtain moments of inertia of net areas for various sizes and numbers of rivets holes, multiply the moment of inertia of the gross area as found in the table by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as ⅛″ larger than the diameter of the rivet.)

Size of Rivet and Number of Rivet Holes in each Angle.	Percentage.
[illegible]	[illegible]
¾″ rivet—2—⅞″ holes	76.4%
⅞″ rivet—1—1″ hole	86.6%
⅞″ rivet—2—1″ holes	73.2%

B.B. Angles.	Thickness of Angles.										
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16
12	225	266	306	341	378	413	446	476	506	537	564
½	249	294	338	378	418	457	495	528	562	596	627
13	273	323	371	415	459	502	544	581	619	656	690
½	299	354	407	455	504	552	598	639	681	722	760
14	326	385	443	496	550	602	652	698	743	788	830
½	355	419	483	541	600	656	711	762	811	861	907
15	384	454	523	586	650	711	771	826	880	934	984
½	415	491	566	634	704	771	836	895	954	1013	1068
16	447	529	609	683	758	831	901	965	1029	1093	1153
½	481	569	656	736	817	895	971	1041	1111	1179	1244
17	515	610	703	789	876	960	1041	1117	1193	1266	1335
½	551	653	753	845	939	1029	1116	1198	1279	1359	1433
18	588	697	803	902	1002	1099	1192	1280	1366	1452	1532
½	627	744	857	963	1070	1173	1273	1367	1459	1552	1637
19	667	791	911	1024	1138	1248	1355	1454	1553	1652	1743
½	708	840	968	1088	1210	1327	1441	1547	1652	1758	1855
20	750	890	1025	1153	1282	1407	1527	1641	1752	1864	1968
½	794	942	1086	1222	1358	1491	1619	1740	1859	1977	2087
21	839	995	1147	1291	1435	1575	1711	1839	1966	2090	2207
½	886	1051	1211	1363	1516	1664	1808	1943	2077	2210	2334
22	933	1107	1276	1436	1597	1754	1905	2048	2189	2330	2461
½	982	1165	1344	1512	1683	1848	2008	2158	2307	2456	2593
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
½	1084	1286	1483	1670	1859	2041	2218	2385	2550	2715	2870
24	1136	1348	1555	1751	1949	2141	2326	2502	2675	2848	3011
½	1191	1413	1630	1835	2043	2244	2439	2624	2806	2988	3159
25	1247	1478	1705	1920	2138	2348	2553	2746	2937	3128	3307
½	1303	1546	1783	2009	2237	2457	2671	2874	3074	3274	3462
26	1359	1614	1862	2098	2336	2566	2790	3003	3212	3421	3617
½	1419	1685	1944	2191	2439	2680	2914	3136	3355	3574	3779

TABLE 17 (*Continued*)

MOMENTS OF INERTIA FOR 4—5″ x 3½″ ANGLES (LONG LEG AGAINST WEB, 2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B.B. Angles.	Thickness of Angles.										
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16
27	1479	1756	2026	2284	2543	2794	3038	3270	3499	3727	3941
27½	1541	1830	2111	2380	2649	2913	3167	3410	3649	3887	4110
28	1603	1904	2197	2476	2756	3032	3297	3550	3799	4047	4280
28½	1668	1981	2286	2577	2869	3155	3432	3695	3954	4213	4456
29	1733	2058	2375	2678	2983	3279	3567	3840	4110	4380	4633
29½	1800	2138	2467	2782	3100	3407	3707	3991	4273	4553	4817
30	1868	2218	2560	2887	3217	3536	3847	4143	4436	4726	5001
30½	1938	2301	2656	2996	3338	3670	3992	4300	4604	4905	5191
31	2008	2385	2753	3105	3459	3804	4138	4457	4772	5085	5381
31½	2080	2471	2852	3217	3585	3942	4289	4620	4946	5271	5578
32	2152	2557	2952	3330	3711	4081	4440	4784	5121	5458	5776
32½	2227	2647	3055	3447	3841	4224	4596	4952	5303	5651	5981
33	2303	2737	3159	3564	3971	4368	4753	5120	5485	5845	6186
33½	2380	2829	3265	3684	4106	4514	4914	5297	5671	6044	6397
34	2458	2921	3372	3805	4241	4660	5076	5474	5858	6244	6609
34½	2539	3016	3482	3930	4380	4815	5243	5653	6052	6451	6828
35	2620	3111	3592	4055	4520	4971	5410	5833	6247	6658	7048
35½	2702	3210	3706	4183	4663	5129	5582	6018	6446	6870	7274
36	2784	3309	3820	4312	4807	5287	5755	6204	6646	7083	7500
36½	2869	3410	3937	4445	4954	5450	5933	6396	6851	7303	7733
37½	3042	3616	4176	4715	5255	5782	6294	6786	7269	7750	8206
38½	3220	3828	4421	4992	5565	6123	6667	7187	7699	8210	8693
39½	3404	4046	4673	5277	5884	6474	7050	7601	8142	8683	9195
40½	3592	4271	4933	5571	6212	6836	7443	8026	8599	9170	9712
41½	3786	4502	5199	5872	6548	7206	7847	8463	9067	9670	10241
42½	3985	4738	5473	6181	6893	7586	8262	8911	9547	10183	10786
43½	4189	4981	5754	6499	7248	7976	8687	9370	10040	10709	11344
44½	4398	5229	6042	6825	7611	8377	9123	9840	10546	11249	11916
45½	4612	5484	6336	7158	7983	8787	9570	10323	11064	11803	12502
46½	4831	5745	6637	7499	8364	9207	10028	10818	11595	12369	13103
47½	5055	6012	6946	7849	8755	9637	10497	11323	12140	12949	13719
48½	5285	6285	7262	8206	9153	10076	10976	11842	12695	13543	14348
49½	5519	6564	7586	8572	9561	10526	11466	12374	13263	14150	14992
50½	5759	6850	7916	8945	9978	10986	11968	12914	13844	14771	15651
51½	6004	7042	8252	9327	10405	11455	12480	13466	14438	15405	16322
52½	6254	7440	8597	9716	10840	11935	13002	14031	15044	16052	17008
53½	6509	7743	8948	10113	11282	12424	13535	14607	15662	16711	17709
54½	6770	8053	9306	10517	11734	12922	14078	15095	16291	17383	18422
55½	7034	8369	9672	10931	12198	13431	14632	15795	16934	18072	19151
56½	7305	8691	10043	11352	12668	13949	15199	16407	17590	18773	19893
57½	7580	9018	10422	11782	13146	14478	15777	17029	18259	19485	20650
58½	7861	9353	10809	12220	13636	15017	16365	17663	18939	20213	21424
59½	8147	9694	11202	12665	14134	15565	16962	18308	19633	20953	22207
60	8291	9865	11400	12890	14384	15842	17262	18636	19984	21325	22603
60½	8437	10037	11601	13120	14639	16121	17568	18966	20339	21708	23006

TABLE 18

MOMENTS OF INERTIA FOR 4—5″ x 3½″ ANGLES, SHORT LEG AGAINST WEB (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area. To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found in the table by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.)

Size of Rivet and Number of Rivet Holes in each Angle.	Percentage.
$\frac{3}{4}$″ rivet—1—$\frac{7}{8}$″ hole	88.1%
$\frac{3}{4}$″ rivet—2—$\frac{7}{8}$″ holes	76.4%
$\frac{7}{8}$″ rivet—1—1″ hole	86.6%
$\frac{7}{8}$″ rivet—2—1″ holes	73.2%

B. B. Angles.	Thickness of Angles.										
	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$
12	284	335	385	431	478	522	566	604	644	682	718
12½	311	368	422	473	525	574	622	664	709	752	791
13	339	401	460	516	573	626	679	726	775	822	865
13½	369	437	501	563	623	683	741	792	846	898	946
14	400	473	543	610	677	741	804	860	918	974	1027
14½	432	517	588	660	733	803	871	931	996	1057	1114
15	465	551	633	711	790	865	939	1006	1074	1141	1202
15½	500	593	681	765	851	932	1012	1083	1158	1230	1297
16	536	635	730	820	912	999	1085	1163	1242	1320	1392
16½	574	680	782	879	977	1071	1163	1247	1332	1415	1494
17	613	725	834	938	1043	1144	1241	1332	1423	1511	1596
17½	653	773	889	1001	1113	1220	1325	1422	1520	1615	1704
18	694	822	945	1064	1183	1297	1409	1512	1617	1719	1812
18½	736	873	1004	1130	1257	1379	1498	1608	1719	1828	1929
19	779	924	1063	1197	1331	1461	1587	1705	1822	1938	2046
19½	824	978	1125	1268	1410	1547	1681	1806	1931	2054	2169
20	870	1032	1188	1339	1489	1634	1776	1908	2040	2171	2293
20½	918	1089	1254	1413	1572	1726	1876	2015	2155	2294	2423
21	967	1147	1320	1488	1656	1818	1976	2123	2271	2417	2553
21½	1017	1207	1389	1566	1746	1914	2081	2235	2392	2541	2690
22	1068	1268	1459	1645	1837	2011	2186	2350	2514	2666	2828
22½	1121	1331	1532	1728	1926	2112	2297	2468	2642	2807	2972
23	1175	1394	1606	1811	2016	2214	2408	2589	2770	2949	3117
23½	1231	1461	1682	1897	2112	2320	2524	2714	2904	3092	3268
24	1287	1528	1759	1984	2209	2427	2640	2839	3039	3235	3420
24½	1345	1596	1839	2075	2310	2538	2761	2970	3179	3384	3578
25	1404	1666	1919	2166	2412	2650	2882	3101	3319	3534	3737
25½	1464	1738	2002	2260	2517	2766	3008	3237	3465	3690	3903
26	1525	1811	2086	2355	2623	2883	3135	3374	3612	3847	4069
26½	1588	1887	2173	2454	2733	3004	3267	3513	3765	4015	4242

TABLE 18 (*Continued*)

MOMENTS OF INERTIA FOR 4—5″ x 3½″ ANGLES, SHORT LEG AGAINST WEB (2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B. B. Angles.	Thickness of Angles.										
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16
27	1652	1963	2261	2553	2843	3125	3400	3660	3918	4174	4415
27½	1718	2041	2352	2655	2958	3246	3537	3808	4077	4344	4595
28	1785	2120	2443	2758	3073	3378	3675	3957	4237	4514	4775
28½	1853	2201	2537	2864	3192	3509	3817	4111	4402	4690	4961
29	1922	2283	2631	2971	3311	3640	3960	4265	4567	4866	5147
29½	1993	2368	2729	3082	3434	3776	4108	4425	4739	5049	5342
30	2065	2453	2827	3193	3558	3912	4257	4585	4911	5233	5537
30½	2138	2540	2928	3308	3686	4053	4410	4751	5088	5422	5737
31	2212	2628	3029	3423	3814	4194	4564	4917	5266	5612	5942
31½	2288	2719	3134	3541	3946	4340	4723	5088	5450	5808	6149
32	2364	2810	3239	3660	4079	4486	4883	5260	5634	6004	6359
32½	2442	2903	3347	3782	4216	4637	5047	5438	5825	6208	6571
33	2522	2997	3455	3905	4353	4788	5212	5616	6016	6412	6787
33½	2603	3094	3567	4031	4494	4943	5381	5798	6212	6621	7009
34	2685	3191	3679	4158	4635	5099	5550	5981	6408	6831	7232
34½	2768	3291	3794	4289	4781	5259	5725	6170	6611	7048	7462
35	2853	3392	3910	4420	4928	5420	5901	6360	6814	7265	7692
35½	2939	3494	4029	4554	5078	5586	6081	6555	7023	7488	7928
36	3027	3597	4148	4689	5228	5752	6262	6750	7232	7711	8165

TABLE 19

MOMENTS OF INERTIA 4—6″ x 4″ ANGLES (2 IN EACH FLANGE 6″ LEG AGAINST WEB), FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area. To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found in the table by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.)

Size of Rivet and Number of Rivet Holes in each Angle.	Percentage.
$\frac{3}{4}$″ rivet—1—$\frac{7}{8}$″ hole	90.7%
$\frac{3}{4}$″ rivet—2—$\frac{7}{8}$″ holes	81.1%
$\frac{7}{8}$″ rivet—1—1″ hole	89.2%
$\frac{7}{8}$″ rivet—2—1″ holes	78.4%
1″ rivet—1—$1\frac{1}{8}$″ hole	88.0%
1″ rivet—2—$1\frac{1}{8}$″ holes	75.8%

B.B. Angles.	Thickness of Angles.										
	$\frac{3}{8}$″	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″	$\frac{7}{8}$″	$\frac{15}{16}$″	1″
16	583	673	756	839	920	996	1071	1110	1216	1285	1347
$\frac{1}{2}$	629	726	815	905	992	1075	1156	1200	1314	1388	1455
17	675	779	875	972	1065	1154	1242	1291	1412	1492	1566
$\frac{1}{2}$	724	836	939	1043	1144	1240	1334	1389	1518	1604	1684
18	774	893	1004	1115	1223	1326	1427	1487	1624	1717	1802
$\frac{1}{2}$	826	954	1073	1192	1307	1418	1527	1592	1738	1838	1929
19	879	1015	1142	1269	1392	1510	1627	1697	1852	1959	2057
$\frac{1}{2}$	935	1080	1215	1351	1482	1608	1733	1810	1974	2088	2193
20	992	1145	1289	1433	1573	1707	1839	1923	2096	2218	2330
$\frac{1}{2}$	1052	1215	1367	1520	1669	1812	1952	2043	2225	2355	2475
21	1112	1284	1446	1608	1766	1918	2066	2163	2355	2493	2621
$\frac{1}{2}$	1175	1358	1529	1701	1868	2029	2186	2291	2493	2639	2775
22	1239	1432	1613	1794	1970	2141	2307	2419	2631	2786	2930
$\frac{1}{2}$	1304	1509	1700	1892	2078	2258	2434	2554	2777	2941	3093
23	1370	1587	1788	1990	2186	2376	2562	2689	2923	3096	3257
$\frac{1}{2}$	1442	1669	1881	2093	2300	2500	2696	2832	3077	3159	3429
24	1515	1752	1974	2197	2414	2624	2830	2975	3231	3423	3602
$\frac{1}{2}$	1589	1838	2071	2306	2534	2755	2971	3125	3393	3595	3783
25	1664	1924	2169	2415	2654	2886	3112	3275	3555	3767	3964
$\frac{1}{2}$	1740	2014	2271	2529	2779	3023	3260	3432	3724	3947	4155
26	1816	2105	2373	2643	2905	3160	3408	3590	3894	4127	4346
$\frac{1}{2}$	1900	2199	2480	2762	3036	3303	3563	3755	4072	4316	4545
27	1984	2294	2587	2882	3168	3447	3718	3921	4250	4505	4744
$\frac{1}{2}$	2069	2393	2699	3006	3305	3596	3880	4093	4436	4702	4952
28	2154	2492	2811	3131	3442	3746	4042	4246	4622	4900	5161
$\frac{1}{2}$	2242	2595	2927	3260	3585	3902	4211	4445	4815	5105	5379
29	2330	2698	3043	3390	3729	4059	4380	4625	5009	5311	5597
$\frac{1}{2}$	2423	2805	3164	3525	3878	4221	4556	4813	5211	5525	5822
30	2517	2912	3286	3661	4028	4384	4732	5001	5413	5740	6048
$\frac{1}{2}$	2613	3023	3412	3801	4182	4553	4914	5196	5623	5962	6284

TABLE 19 (*Continued*)

MOMENTS OF INERTIA 4—6″ x 4″ ANGLES (2 IN EACH FLANGE 6″ LEG AGAINST WEB), FOR VARIOUS DISTANCES BACK TO BACK

B.B. Angles.	Thickness of Angles.										
	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
31	2710	3135	3538	3942	4337	4723	5097	5391	5833	6185	6520
½	2809	3250	3668	4088	4497	4898	5287	5593	6051	6417	6764
32	2908	3366	3799	4234	4658	5074	5478	5796	6269	6649	7009
½	3012	3486	3934	4385	4825	5255	5674	6005	6494	6888	7262
33	3116	3606	4070	4536	4993	5437	5871	6215	6719	7127	7516
½	3222	3729	4210	4693	5164	5625	6074	6432	6953	7376	7778
34	3329	3853	4351	4850	5336	5814	6278	6650	7188	7625	8041
½	3440	3981	4496	5012	5515	6009	6488	6875	7429	7882	8312
35	3551	4110	4641	5174	5694	6204	6699	7100	7671	8139	8583
½	3664	4242	4790	5341	5878	6404	6916	7332	7921	8404	8864
36	3778	4374	4940	5508	6062	6605	7134	7565	8171	8669	9145
½	3896	4511	5094	5680	6252	6813	7358	7804	8428	8944	9434
37	4136	4788	5408	6031	6638	7235	7814	8291	8952	9500	10022
38	4384	5074	5731	6392	7037	7670	8284	8793	9492	10073	10622
39	4637	5368	6064	6763	7446	8117	8767	9310	10047	10662	11246
40	4896	5670	6406	7145	7867	8577	9264	9842	10619	11270	11894
41	5164	5981	6758	7538	8300	9049	9776	10389	11205	11894	12555
42	5439	6299	7119	7941	8745	9535	10301	10950	11808	12534	13232
43	5721	6627	7490	8356	9201	10034	10841	11527	12429	13193	13927
44	6011	6963	7871	8780	9669	10545	11394	12118	13063	13868	14642
45	6308	7308	8261	9214	10150	11068	11961	12724	13714	14560	15373
46	6613	7660	8660	9660	10641	11605	12541	13347	14381	15270	16123
47	6924	8021	9068	10116	11143	12155	13135	13984	15064	15997	16891
48	7242	8390	9486	10583	11658	12717	13743	14635	15766	16741	17677
49	7569	8768	9914	11061	12186	13293	14365	15300	16481	17501	18480
50	7902	9154	10350	11551	12724	13882	15002	15981	17211	18278	19303
51	8242	9548	10796	12049	13273	14482	15652	16679	17958	19073	20142
52	8589	9951	11253	12559	13837	15096	16316	17391	18723	19884	21000
53	8944	10363	11721	13081	14412	15724	16996	18117	19502	20710	21876
54	9306	10782	12196	13611	14996	16362	17686	18857	20296	21555	22769
55	9674	11109	12679	14152	15592	17013	18392	19613	21108	22420	23680
56	10051	11647	13173	14705	16202	17679	19113	20384	21935	23299	24610
57	10434	12092	13677	15265	16822	18358	19844	21169	22777	24193	25557
58	10825	12545	14190	15838	17454	19050	20591	21970	23637	25106	26523
59	11223	13007	14714	16422	18099	19754	21353	22785	24512	26037	27508
60	11628	13477	15246	17016	18754	20468	22128	23615	25402	26983	28510
61	12041	13954	15786	17621	19420	21197	22917	24461	26310	27948	29530
62	12461	14442	16339	18237	20100	21940	23721	25321	27235	28932	30568
63	12887	14938	16899	18863	20791	22695	24537	26198	28174	29931	31623
64	13321	15441	17468	19501	21492	23463	25368	27089	29124	30946	32696
65	13762	15953	18049	20150	22206	24243	26213	27996	30094	31977	33790
66	14210	16473	18640	20807	22934	25035	27072	28917	31089	33025	34901
67	14666	17002	19239	21476	23672	25841	27946	29852	32095	34090	36031
68	15130	17538	19845	22156	24422	26662	28832	30804	33110	35175	37177
69	15601	18083	20464	22846	25183	27494	29731	31768	34141	36278	38338
70	16079	18641	21093	23546	25954	28339	30646	32744	35192	37400	39519
71	16563	19202	21730	24259	26742	29198	31572	33741	36261	38529	40720
72	17054	19772	22375	24977	27534	30067	32518	34955	37341	39677	41933

TABLE 20

MOMENTS OF INERTIA FOR 4—6″ x 4″ ANGLES (SHORT LEG AGAINST WEB, 2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

The tables are made for gross area. To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found in the tables by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.)

Size of Rivet and Number of Rivet Holes in each Angle.	Percentage.
$\frac{3}{4}$″ rivet—1—$\frac{7}{8}$″ hole	90.7%
$\frac{3}{4}$″ rivet—2—$\frac{7}{8}$″ holes	81.1%
$\frac{7}{8}$″ rivet—1—1″ hole	89.2%
$\frac{7}{8}$″ rivet—2—1″ holes	78.4%
1″ rivet—1—1$\frac{1}{8}$″ hole	88.0%
1″ rivet—2—1$\frac{1}{8}$″ holes	75.8%

B.B. Angles.	Thickness of Angles.										
	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
12	390	448	502	557	609	658	707	754	800	844	883
$\frac{1}{2}$	428	492	552	612	670	724	779	831	882	931	974
13	466	536	602	668	731	791	851	908	964	1018	1066
$\frac{1}{2}$	508	585	656	729	798	864	929	992	1054	1113	1166
14	550	634	711	790	866	937	1008	1077	1144	1209	1267
$\frac{1}{2}$	595	686	770	856	938	1016	1093	1169	1242	1312	1376
15	641	739	830	923	1011	1095	1179	1261	1340	1416	1486
$\frac{1}{2}$	690	796	894	994	1090	1181	1271	1360	1446	1528	1604
16	740	853	959	1066	1169	1267	1364	1460	1552	1641	1723
$\frac{1}{2}$	792	914	1028	1143	1253	1359	1464	1566	1666	1762	1850
17	845	975	1097	1220	1338	1451	1564	1673	1780	1883	1977
$\frac{1}{2}$	891	1040	1170	1302	1428	1549	1670	1787	1902	2012	2113
18	958	1106	1244	1384	1519	1648	1776	1902	2024	2142	2250
$\frac{1}{2}$	1018	1175	1322	1471	1615	1753	1889	2023	2153	2279	2395
19	1078	1244	1401	1559	1712	1858	2003	2145	2283	2417	2541
$\frac{1}{2}$	1141	1318	1484	1652	1814	1969	2123	2274	2421	2563	2695
20	1205	1392	1568	1745	1916	2081	2243	2404	2559	2710	2850
$\frac{1}{2}$	1272	1470	1656	1843	2024	2199	2371	2540	2705	2864	3013
21	1340	1548	1744	1941	2132	2317	2499	2677	2851	3019	3177
$\frac{1}{2}$	1410	1630	1836	2044	2246	2441	2633	2821	3005	3183	3349
22	1481	1712	1929	2148	2360	2565	2767	2965	3159	3347	3521
$\frac{1}{2}$	1555	1798	2026	2256	2480	2696	2908	3117	3321	3518	3702
23	1630	1884	2124	2365	2600	2827	3049	3269	3483	3690	3884
$\frac{1}{2}$	1708	1974	2226	2479	2725	2964	3197	3428	3654	3870	4074
24	1786	2065	2328	2594	2851	3101	3346	3587	3826	4051	4265
$\frac{1}{2}$	1868	2159	2435	2713	2982	3244	3500	3753	4002	4240	4464
25	1950	2254	2542	2832	3114	3388	3655	3919	4178	4429	4664
$\frac{1}{2}$	2035	2353	2653	2955	3251	3537	3817	4094	4364	4626	4872
26	2120	2452	2765	3078	3388	3687	3980	4269	4550	4824	5081
$\frac{1}{2}$	2209	2555	2882	3209	3531	3843	4148	4450	4744	5029	5298

TABLE 20 (*Continued*)

MOMENTS OF INERTIA FOR 4—6″ x 4″ ANGLES (SHORT LEG AGAINST WEB, 2 IN EACH FLANGE) FOR VARIOUS DISTANCES BACK TO BACK

B.B. Angles.	Thickness of Angles.										
	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
27	2298	2658	2999	3341	3675	4000	4317	4631	4938	5235	5516
$\frac{1}{2}$	2390	2765	3120	3476	3824	4163	4493	4820	5139	5449	5742
28	2483	2872	3241	3612	3973	4326	4669	5010	5341	5664	5969
$\frac{1}{2}$	2579	2983	3367	3752	4128	4494	4851	5206	5551	5887	6204
29	2675	3095	3493	3893	4283	4663	5034	5403	5761	6110	6440
$\frac{1}{2}$	2775	3210	3623	4039	4443	4839	5224	5606	5978	6341	6684
30	2875	3326	3754	4185	4604	5015	5414	5810	6195	6572	6929
$\frac{1}{2}$	2978	3446	3890	4336	4771	5196	5610	6021	6421	6811	7182
31	3082	3566	4026	4487	4938	5378	5809	6233	6648	7051	7435
$\frac{1}{2}$	3188	3689	4166	4644	5110	5566	6010	6452	6881	7300	7698
32	3295	3813	4306	4801	5283	5755	6214	6671	7115	7549	7961
$\frac{1}{2}$	3406	3941	4450	4963	5461	5950	6425	6897	7357	7806	8232
33	3517	4070	4595	5125	5640	6145	6636	7124	7599	8063	8503
$\frac{1}{2}$	3631	4202	4745	5292	5824	6346	6853	7357	7848	8328	8784
34	3745	4334	4896	5459	6008	6547	7071	7591	8098	8593	9065
$\frac{1}{2}$	3862	4471	5050	5631	6198	6754	7295	7832	8356	8867	9354
35	3980	4608	5205	5804	6388	6962	7519	8074	8614	9141	9643
$\frac{1}{2}$	4101	4748	5364	5981	6584	7176	7750	8323	8879	9423	9941
36	4223	4889	5523	6159	6781	7390	7982	8572	9145	9706	10240

TABLE 21

MOMENTS OF INERTIA 4—8″ x 6″ ANGLES (2 IN EACH FLANGE) (8″ LEG AGAINST WEB) FOR VARIOUS DISTANCES BACK TO BACK.

To obtain the moment of inertia of 4—8″ x 6″ angles with the 8″ leg against the web multiply the moment of inertia of 4—8″ x 8″ angles of the given thickness and distance back to back as found in Table 16 by the percentage corresponding to the depth used and obtained from the diagram Table 21.

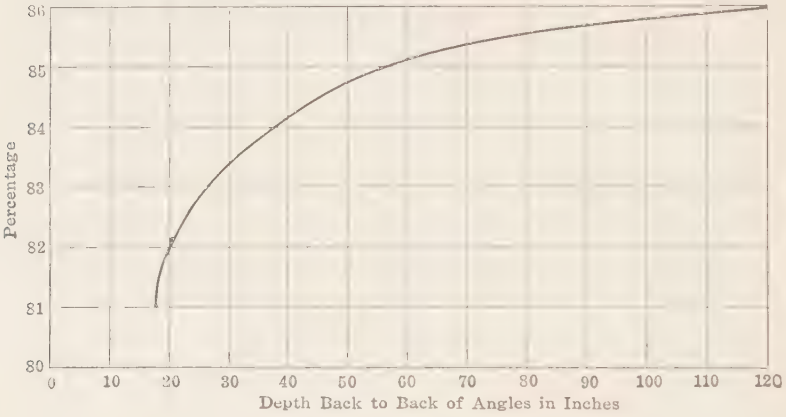


TABLE 22

8″ x 6″ ANGLES

To obtain moments of inertia of net areas for various sizes and numbers of rivet holes, multiply the moment of inertia of the gross area as found above by the percentage corresponding to the size and number of rivet holes in each angle. (Rivet holes are computed as $\frac{1}{8}$″ larger than the diameter of the rivet.)

Size of Rivet and Number of Rivet Holes in each Angle.	Percentage.
$\frac{7}{8}$″ rivet—1—1″ hole	92.5
$\frac{7}{8}$″ rivet—2—1″ holes	84.9
1″ rivet—1—$1\frac{1}{8}$″ hole	91.6
1″ rivet—2—$1\frac{1}{8}$″ holes	83.1
$1\frac{1}{8}$″ rivet—2—$1\frac{1}{4}$″ holes	81.1
$\frac{7}{8}$″ rivet—3—1″ holes	77.4
1″ rivet—3—$1\frac{1}{8}$″ holes	74.5

TABLE 23—MOMENTS OF INERTIA OF COVER PLATES (INCHES⁴) FOR PLATES 10″ WIDE

Depth in Clear Between Plates.	Thickness of Plates.																				
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1″	1 1/16	1 1/8	1 3/16	1 1/4	1 5/16	1 3/8	1 7/16	1 1/2	1 9/16
12	238	287	339	391	444	498	553	611	668	726	785	845	906	968	1032	1099	1164	1231	1295	1370	1441
12½	247	311	367	425	480	535	597	660	721	785	847	911	977	1043	1111	1184	1253	1325	1400	1474	1549
13	257	336	395	456	517	580	643	710	776	843	911	980	1050	1122	1194	1271	1346	1422	1502	1581	1661
13½	268	361	425	490	556	623	691	763	833	905	978	1051	1126	1202	1280	1362	1441	1522	1607	1691	1777
14	278	388	450	525	596	668	741	817	892	968	1047	1126	1206	1288	1371	1455	1542	1629	1717	1807	1898
14½	313	414	489	563	638	716	793	874	953	1036	1119	1203	1289	1376	1464	1555	1645	1737	1831	1926	2023
15	367	443	522	601	681	763	846	932	1017	1104	1192	1281	1373	1465	1558	1654	1751	1847	1946	2047	2149
15½	391	473	556	640	726	815	901	992	1082	1174	1268	1363	1460	1557	1656	1758	1859	1962	2067	2173	2281
16	416	503	591	681	773	864	957	1055	1150	1245	1344	1445	1549	1654	1761	1864	1972	2080	2191	2303	2416
16½	442	534	628	725	820	917	1016	1118	1219	1322	1427	1534	1643	1641	1869	1974	2087	2201	2318	2436	2555
17	469	566	665	766	868	970	1075	1183	1291	1399	1510	1622	1735	1850	1966	2086	2205	2326	2449	2573	2698
17½	496	599	701	810	918	1027	1137	1251	1364	1478	1595	1715	1832	1953	2076	2202	2327	2457	2583	2713	2844
18	525	633	741	856	969	1084	1200	1320	1439	1560	1683	1807	1932	2059	2188	2321	2452	2586	2724	2855	2993
18½	554	668	782	903	1022	1143	1266	1392	1517	1644	1775	1903	2034	2169	2304	2443	2580	2721	2865	3005	3149
19	583	704	825	951	1076	1203	1332	1466	1596	1730	1866	2004	2137	2280	2424	2566	2714	2859	3008	3158	3308
19½	614	741	870	1000	1134	1266	1401	1540	1677	1818	1959	2104	2246	2394	2544	2695	2847	3001	3160	3318	3474
20	645	779	911	1051	1186	1329	1470	1615	1759	1905	2057	2206	2357	2511	2666	2827	2986	3146	3309	3475	3637
20½	677	818	960	1103	1246	1395	1543	1695	1847	2001	2158	2314	2471	2633	2795	2961	3127	3295	3465	3636	3808
21	711	857	1006	1156	1306	1461	1616	1776	1935	2095	2258	2422	2586	2755	2925	3097	3272	3447	3624	3804	3982
21½	744	898	1053	1210	1368	1530	1692	1858	2024	2192	2361	2533	2705	2884	3058	3239	3420	3609	3787	3973	4160
22	779	939	1102	1266	1430	1599	1769	1944	2116	2291	2468	2647	2826	3010	3197	3383	3571	3756	3954	4148	4342
22½	811	982	1152	1323	1495	1671	1848	2029	2210	2390	2577	2763	2950	3142	3334	3530	3726	3922	4124	4326	4529
23	850	1025	1202	1381	1560	1741	1928	2117	2305	2490	2688	2882	3076	3276	3475	3678	3884	4090	4298	4508	4717
23½	887	1067	1251	1440	1627	1819	2011	2207	2403	2599	2801	3006	3205	3415	3620	3831	4045	4257	4475	4696	4912
24	921	1109	1307	1501	1695	1895	2094	2297	2503	2708	2917	3127	3337	3552	3768	3987	4210	4426	4656	4885	5109
24½	963	1159	1361	1564	1765	1973	2180	2391	2605	2817	3035	3253	3472	3699	3920	4147	4377	4606	4841	5076	5311
25	1002	1208	1416	1626	1836	2052	2267	2486	2709	2925	3155	3382	3608	3841	4075	4309	4548	4788	5029	5273	5516
25½	1042	1254	1472	1690	1909	2133	2357	2581	2811	3043	3278	3515	3748	3990	4230	4475	4722	4968	5221	5474	5726
26	1083	1299	1529	1759	1982	2215	2447	2683	2922	3162	3403	3647	3890	4141	4390	4643	4898	5151	5416	5678	5939
26½	1124	1353	1588	1823	2058	2300	2540	2785	3032	3278	3531	3783	4036	4290	4554	4816	5081	5344	5615	5886	6156
27	1166	1406	1647	1891	2134	2385	2634	2887	3142	3395	3660	3922	4183	4451	4719	4990	5265	5540	5818	6098	6377
27½	1209	1455	1708	1960	2213	2467	2731	2993	3258	3521	3792	4063	4333	4611	4885	5168	5451	5734	6024	6313	6602
28	1253	1504	1769	2031	2292	2560	2828	3099	3374	3649	3927	4207	4486	4773	5060	5349	5644	5931	6234	6533	6841
28½	1298	1561	1832	2103	2373	2651	2928	3208	3491	3774	4064	4303	4648	4938	5235	5536	5837	6138	6448	6756	7069
29	1343	1618	1896	2176	2455	2742	3029	3318	3613	3900	4203	4502	4800	5106	5412	5724	6035	6348	6665	6983	7301
29½	1389	1671	1961	2250	2539	2835	3132	3433	3734	4035	4345	4653	4961	5278	5593	5913	6235	6556	6885	7215	7543

MOMENTS OF INERTIA OF COVER PLATES (INCHES^4) FOR PLATES 10″ WIDE—TABLE 23 (*Continued*)

Depth in Clear Between Plates.	Thickness of Plates.																				
	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1″	1 1/16	1 1/8	1 3/16	1 1/4	1 5/16	1 3/8	1 7/16	1 1/2	1 9/16
30	1457	1724	2027	2366	2624	2930	3256	3548	3859	4172	4489	4807	5125	5452	5776	6105	6439	6766	7109	7448	7789
30½	1484	1785	2095	2408	2711	3048	3443	3667	3985	4306	4635	4963	5293	5629	5964	6303	6646	6987	7337	7686	8038
31	1513	1840	2163	2481	2799	3126	3470	3779	4113	4440	4783	5122	5463	5807	6153	6502	6857	7211	7568	7928	8290
31½	1542	1896	2232	2560	2888	3226	3591	3900	4246	4584	4934	5283	5635	5989	6346	6705	7070	7432	7803	8173	8545
32	1572	1959	2302	2644	2982	3326	3672	4021	4376	4730	5087	5447	5809	6173	6541	6911	7285	7656	8042	8423	8803
32½	1605	2024	2374	2723	3075	3429	3786	4145	4510	1892	5243	5613	5985	6360	6740	7121	7508	7891	8284	8676	9069
33	1755	2089	2446	2806	3168	3532	3900	4270	4647	5015	5401	5782	6162	6552	6941	7333	7731	8129	8530	8933	9339
33½	1787	2140	2512	2883	3252	3627	4017	4398	4785	5168	5563	5955	6346	6745	7146	7549	7958	8368	8779	9193	9612
34	1841	2209	2595	2976	3356	3746	4135	4527	4925	5324	5724	6127	6533	6941	7353	7767	8188	8609	9032	9458	9887
34½	1895	2278	2671	3068	3456	3855	4256	4659	5068	5478	5889	6303	6722	7141	7564	7990	8421	8854	9289	9726	10167
35	1949	2347	2748	3151	3557	3965	4377	4791	5212	5633	6056	6482	6911	7343	7778	8214	8658	9102	9549	9998	10450
35½	2005	2415	2826	3240	3659	4078	4501	4927	5358	5791	6226	6665	7104	7548	7993	8442	8898	9355	9813	10273	10738
36	2061	2482	2907	3331	3760	4192	4626	5063	5507	5951	6398	6847	7299	7755	8211	8673	9142	9611	10080	10553	11029
36½	2118	2551	2988	3423	3864	4308	4756	5202	5658	6115	6573	7033	7498	7965	8435	8910	9388	9866	10351	10836	11325
37	2237	2692	3150	3610	4075	4542	5012	5488	5965	6446	6928	7413	7902	8395	8889	9391	9891	10396	10903	11413	11927
37½	2355	2836	3317	3803	4292	4783	5279	5781	6281	6786	7293	7805	8317	8841	9349	9882	10407	10937	11470	12006	12542
38	2475	2981	4488	4000	4514	5031	5546	6076	6604	7134	7667	8205	8745	9286	9830	10385	10936	11491	12051	12613	13179
38½	2594	3124	3667	4205	4745	5286	5834	6385	6936	7491	8051	8614	9179	9748	10320	10902	11478	12061	12647	13236	13828
39	2713	3268	3848	4410	4976	5545	6117	6699	7275	7858	8444	9033	9625	10222	10820	11430	12033	12644	13257	13873	14493
39½	2865	3448	4034	4625	5215	5812	6410	7025	7628	8234	8847	9463	10085	10708	11331	11971	12602	13237	13881	14526	15174
40	3016	3611	4224	4840	5460	6085	6711	7348	7979	8614	9258	9905	10554	11204	11859	12524	13183	13851	14520	15193	15870
40½	3169	3777	4418	5065	5711	6364	7019	7684	8349	9011	9680	10353	11030	11712	12398	13089	13778	14475	15174	15876	16582
41	3323	3948	4617	5290	5967	6649	7335	8026	8715	9412	10111	10813	11519	12230	12944	13667	14387	15112	15841	16573	17310
41½	3425	4122	4821	5523	6229	6940	7654	8377	9097	9822	10551	11283	12020	12761	13505	14259	15007	16091	16523	17280	18053
42	3574	4299	5028	5760	6497	7238	7982	8735	9484	10240	11000	11763	12530	13302	14077	14862	15641	16766	17219	18013	18812
42½	3724	4480	5240	6003	6770	7542	8314	9094	9881	10669	11459	12253	13052	13855	14660	15477	16288	17452	17930	18755	19586
43	3878	4666	5456	6250	7049	7852	8657	9474	10285	11101	11927	12753	13583	14418	15255	16105	16949	17799	18654	19513	20376
43½	4035	4854	5677	6503	7333	8168	9005	9855	10699	11549	12401	13263	14126	14994	15865	16747	17622	18505	19394	20286	21182
44	4196	5046	5901	6760	7623	8491	9362	10241	11115	12002	12891	13783	14679	15579	16484	17399	18309	19225	20148	21073	22003
44½	4359	5242	6131	7023	7919	8819	9724	10639	11549	12465	13388	14313	15243	16177	17115	18065	19009	19950	20917	21876	22840
45	4526	5442	6365	7290	8220	9154	10092	11042	11986	12930	13894	14853	15847	16786	17767	18741	19722	20708	21699	22693	23688
45½	4696	5646	6604	7563	8527	9496	10477	11451	12432	13418	14399	15403	16398	17407	18415	19435	20445	21469	22496	23526	24556
46	4869	5855	6846	7840	8838	9844	10852	11872	12882	13908	14933	15963	16993	18038	19080	20136	21185	22241	23307	24373	25440
46½	5042	6066	7093	8123	9153	10198	11240	12296	13347	14401	15467	16533	17600	18681	19760	20852	21931	23033	24132	25236	26339
47	5222	6281	7344	8410	9477	10558	11637	12729	13808	14910	16008	17113	18216	19334	20451	21580	22699	23836	24972	26113	27253
47½	5406	6500	7599	8703	9806	10924	12043	13170	14289	15426	16562	17703	18849	19998	21154	22321	23478	24654	25827	27006	28190

Depth in Clear Between Plates.	Thickness of Plates.																					
	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$	$1\frac{3}{16}$	$1\frac{1}{4}$	$1\frac{5}{16}$	$1\frac{3}{8}$	$1\frac{7}{16}$	$1\frac{1}{2}$	$1\frac{9}{16}$
59½	5592	6723	7859	9000	10146	11295	12451	13613	14775	15954	17121	18303	19187	20671	21865	23075	24274	25483	26696	27915	29136	
60½	5780	6951	8121	9303	10487	11674	12868	14075	15274	16483	17696	18913	20136	21363	22595	23835	25079	26332	27578	28836	30097	
61½	5971	7180	8393	9640	10833	12060	13293	14540	15776	17029	18270	19533	20791	22062	23335	24622	25897	27181	28476	29763	31075	
62½	6166	7419	8666	9923	11184	12452	13725	15012	16287	17574	18866	20163	21465	22774	24086	25412	26726	28057	29388	30726	32069	
63½	6364	7651	8943	10240	11542	12851	14162	15489	16805	18134	19465	20803	22146	23495	24847	26211	27572	28942	30311	31690	33078	
64½	6565	7891	9225	10563	11906	13255	14607	15974	17332	18702	20075	21453	22838	24227	25623	27030	28430	29840	31246	32676	34102	
65½	6769	8139	9512	10890	12275	13664	15060	16467	17867	19277	20693	22113	23539	24971	26409	27850	29301	30751	32211	33675	35143	
66½	6977	8387	9803	11223	12649	14081	15519	16967	18411	19861	21321	22783	24253	25729	27207	28700	30183	31680	33181	34686	36198	
67½	7188	8641	10098	11560	13030	14506	15984	17477	18960	20457	21958	23463	24975	26494	28014	29552	31081	32620	34164	35713	37270	
68½	7400	8897	10398	11903	13415	14934	16454	17992	19521	21060	22601	24153	25710	27270	28835	30416	31989	33571	35162	36756	38357	
69½	7618	9157	10701	12250	13807	15367	16932	18513	20086	21674	23259	24853	26454	28058	29671	31296	32933	34572	36175	37813	39459	
70½	7836	9421	11009	12603	14204	15808	17421	19046	20663	22292	23923	25563	27209	28860	30518	32189	33849	35521	37201	38886	40577	
71½	8060	9689	11322	12960	14606	16257	17915	19586	21246	22921	24598	26283	27974	29670	31373	33088	34799	36519	38242	39973	41711	
72½	8286	9961	11638	13323	15014	16710	18413	20128	21839	23559	25282	27013	28751	30472	32241	34006	35761	37527	39298	41076	42861	
73½	8516	10236	11959	13690	15428	17169	18918	20680	22438	24201	25975	27753	29538	31326	33121	34934	36745	38549	40359	42193	44025	
74½	8748	10514	12285	14063	15848	17636	19431	21244	23045	24859	26679	28503	30335	32174	34018	35877	37725	39586	41453	43327	45205	
75½	8982	10796	12616	14440	16272	18111	19955	21814	23662	25523	27390	29263	31142	33030	34923	36828	38725	40637	42551	44473	46401	
76½	9222	11083	12950	14823	16702	18590	20483	22388	24285	26197	28111	30033	31961	33897	35837	37792	39741	41702	43665	45636	47613	
77½	9462	11374	13288	15210	17138	19074	21014	22969	24918	26879	28842	30813	32790	34776	36766	38770	40767	42780	44793	46813	48841	
78½	9708	11669	13632	15603	17580	19566	21556	23560	25557	27570	29583	31603	33632	35669	37708	39763	41810	43879	45934	48006	50085	
79½	9954	11967	13980	16000	18028	20066	22105	24160	26207	28267	30331	32403	34482	36570	38660	40768	42863	44976	47091	49215	51343	
80½	10207	12267	14331	16403	18482	20570	22660	24768	26863	28976	31091	33213	35344	37481	39624	41783	43930	46096	48262	50436	52621	
81½	10460	12573	14688	16810	18949	21078	23221	25380	27527	29693	31859	34033	36216	38403	40597	42808	45010	47226	49447	51672	53908	
82½	10715	12883	15049	17223	19406	21593	23787	25998	28200	30416	32637	34863	37098	39340	41587	43849	46104	48373	50647	52926	55214	
83½	10977	13193	15414	17640	19875	22117	24364	26626	28881	31150	33421	35703	37991	40286	42588	44901	47210	49541	51860	54193	56536	
84½	11241	13510	15781	18063	20351	22646	24948	27264	29570	31894	34220	36553	38896	41244	43599	45971	48330	50708	53087	55476	57872	
85½	11507	13831	16157	18490	20833	23181	25536	27908	30267	32645	35024	37413	39809	42211	44620	47045	49463	51895	54330	56775	59234	
86½	11777	14155	16534	18923	21319	23721	26130	28555	30973	33405	35840	38284	40733	43189	45652	48133	50609	53096	55587	58086	60592	
87½	12050	14482	16917	19360	21810	24266	26729	29211	31686	34173	36664	39163	41669	44181	46703	49239	51769	54311	56858	59413	61975	
88½	12326	14813	17304	19803	22309	24821	27343	29879	32409	34951	37495	40053	42616	45188	47767	50357	52941	55540	58143	60756	63376	
89½	12606	15148	17695	20250	22813	25385	27964	30554	33138	35737	38336	40953	43573	46200	48834	51487	54127	56783	59445	62113	64791	
90½	12888	15487	18090	20703	23323	25950	28584	31237	33877	36533	39195	41863	44541	47225	49912	52620	55325	58049	56779	63486	66221	
91½	13174	15830	18492	21160	23838	26522	29209	31923	34622	37341	40056	42783	45518	48261	51010	53778	56536	59308	62087	64873	67667	
92½	13462	16181	18896	21623	24358	27101	29850	32618	35376	38048	40927	43713	46507	49309	52118	54947	57763	59591	63431	66271	69130	
93½	13753	16525	19304	22090	24884	27686	30495	33324	36140	38968	41808	44658	47507	50370	53236	56125	58999	61891	64788	67695	70608	
94½	14050	16878	19718	22563	25417	28280	31146	34035	36909	39801	42698	45603	48518	51443	54367	57314	60250	63203	66160	69126	72101	

MOMENTS OF INERTIA OF COVER PLATES (INCHES^4) FOR PLATES 10″ WIDE—TABLE 23 *(Continued)*

Depth in Clear Between Plates.	Thickness of Plates.																			
	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$	$1\frac{3}{16}$	$1\frac{1}{4}$	$1\frac{5}{16}$	$1\frac{3}{8}$	$1\frac{7}{16}$	$1\frac{1}{2}$	$1\frac{9}{16}$
95	14149	17258	20455	23740	27113	30574	34123	37759	41483	45294	49192	53178	57250	61410	65657	69991	74413	78922	83518	88202	92973
96	14647	17868	21177	24575	28062	31638	35303	39057	42899	46830	50850	54958	59156	63442	67817	72281	76834	81476	86206	91026	95935
97	15155	18487	21908	25420	29023	32717	36502	40378	44345	48403	52552	56792	61123	65545	70058	74662	79357	84143	89020	93988	99047
98	15674	19116	22649	26275	29993	33803	37705	41699	45785	49963	54233	58595	63049	67595	72233	76963	81785	86699	91705	96803	101994
99	16203	19756	23401	27141	30972	34895	38911	43018	47218	51510	55895	60372	64942	69605	74360	79208	84149	89183	94310	99530	104843
100	16742	20407	24164	28018	31962	35998	40125	44344	48655	53058	57553	62141	66821	71594	76460	81419	86471	91616	96854	102186	107611
101	17292	21069	24937	28906	32964	37112	41352	45683	50105	54619	59225	63923	68713	73596	78571	83639	88800	94053	99400	104840	110373
102	17852	21742	25721	29804	33977	38237	42592	47034	51562	56178	60881	65672	70551	75518	80573	85716	90948	96268	101677	107175	112762
103	18424	22426	26516	30713	35002	39374	43840	48398	53040	57766	62577	67472	72453	77519	82671	87908	93232	98642	104139	109723	115394
104	19008	23122	27323	31633	36038	40523	45098	49763	54518	59362	64295	69318	74430	79631	84922	90302	95772	101331	106980	112719	118548
105	19603	23830	28142	32565	37086	41686	46370	51142	56002	60950	65987	71113	76329	81634	87029	92514	98089	103754	109510	115357	121295
106	20210	24550	28973	33509	38146	42863	47657	52539	57510	62570	67720	72960	78291	83712	89224	94827	100521	106307	112184	118154	124216
107	20828	25282	29816	34466	39218	44053	48960	53954	59035	64204	69462	74809	80246	85774	91393	97103	102905	108799	114786	120866	127039
108	21458	26027	30671	35436	40303	45256	50281	55387	60578	65854	71216	76665	82201	87825	93537	99338	105229	111210	117281	123443	129697
109	22100	26784	31539	36419	41401	46472	51620	56838	62139	67523	72991	78544	84183	89910	95725	101629	107623	113708	119884	126152	132513
110	22753	27554	32420	37416	42513	47702	52973	58303	63715	69209	74787	80449	86197	92031	97952	103961	110059	116247	122525	128894	135355
111	23418	28336	33314	38427	43639	48946	54339	59784	65307	70911	76599	82370	88227	94170	100201	106321	112531	118832	125225	131711	138291
112	24095	29130	34221	39452	44779	50204	55719	61281	66916	72624	78414	84287	90245	96290	102423	108646	114960	121367	127868	134464	141156
113	24784	29937	35141	40491	45933	51476	57114	62794	68542	74358	80242	86206	92251	98380	104595	110898	117292	123779	130361	137040	143818
114	25485	30756	36074	41544	47101	52762	58523	64324	70185	76107	82092	88142	94260	100448	106709	113046	119463	125963	132549	139224	145991
115	26198	31588	37020	42610	48283	54062	59946	65890	71845	77875	83980	90161	96420	102760	109183	115693	122292	128983	135769	142653	149637
116	26923	32432	37979	43690	49479	55376	61383	67471	73522	79661	85886	92194	98586	105066	111636	118299	125058	131916	138876	145941	153113
117	27660	33289	38952	44784	50689	56704	62834	69066	75217	81464	87808	94253	100801	107456	114220	121096	128087	135196	142426	149780	157261
118	28409	34158	39938	45892	51913	58046	64299	70674	76930	83286	89746	96314	102993	109786	116697	123729	130885	138169	145584	153133	160820
119	29170	35040	40937	47013	53151	59402	65778	72295	78661	85127	91697	98376	105167	112074	119101	126252	133530	140939	148482	156163	163985
120	29943	35934	41949	48147	54403	60773	67272	73929	80410	87027	93784	100685	107734	114935	122292	129809	137490	145338	153358	161553	169928

TABLE 24.

Multipliers to use in getting the moments of inertia of the gross or net sections of cover plates of different widths. The moment of inertia of the plates 10 inches wide of the given thickness and clear distance apart should be taken from Table 23 and multiplied by the multiplier corresponding to the width of the plates whose moment of inertia is sought. The result will be the moment of inertia of the given plates in both flanges about an axis midway between the top and bottom flanges.

Width of Plates.	Gross.	Net, Allowing for Holes for Number and Size of Rivets as Shown.						
		2—$\frac{3}{4}$″	2—$\frac{7}{8}$″	2—1″	2—1$\frac{1}{8}$″	4—$\frac{7}{8}$″	4—1″	4—1$\frac{1}{8}$″
8	.80	.625	.600	.575	.550	.....	.....	.....
10	1.00	.825	.800	.775	.750	.....	.....	.....
12	1.20	1.025	1.000	.975	.950	.....	.....	.....
13	1.30	1.125	1.100	1.075	1.050	.....	.....	.....
14	1.40	1.225	1.200	1.175	1.150	1.000	.950	.900
15	1.50	1.325	1.300	1.275	1.250	1.100	1.050	1.000
16	1.60	1.425	1.400	1.375	1.350	1.200	1.150	1.100
18	1.80	1.625	1.600	1.575	1.550	1.400	1.350	1.300
20	2.00	1.825	1.800	1.775	1.750	1.600	1.550	1.500
22	2.20	2.025	2.000	1.975	1.950	1.800	1.750	1.700
24	2.40	2.225	2.200	2.175	2.150	2.000	1.950	1.900
26	2.60	.....	.....	.....	.....	2.200	.....	2.100
28	2.80	.....	.....	.....	.....	2.400	.....	2.300
30	3.00	.....	.....	.....	.....	2.600	.....	2.500

TABLE 25

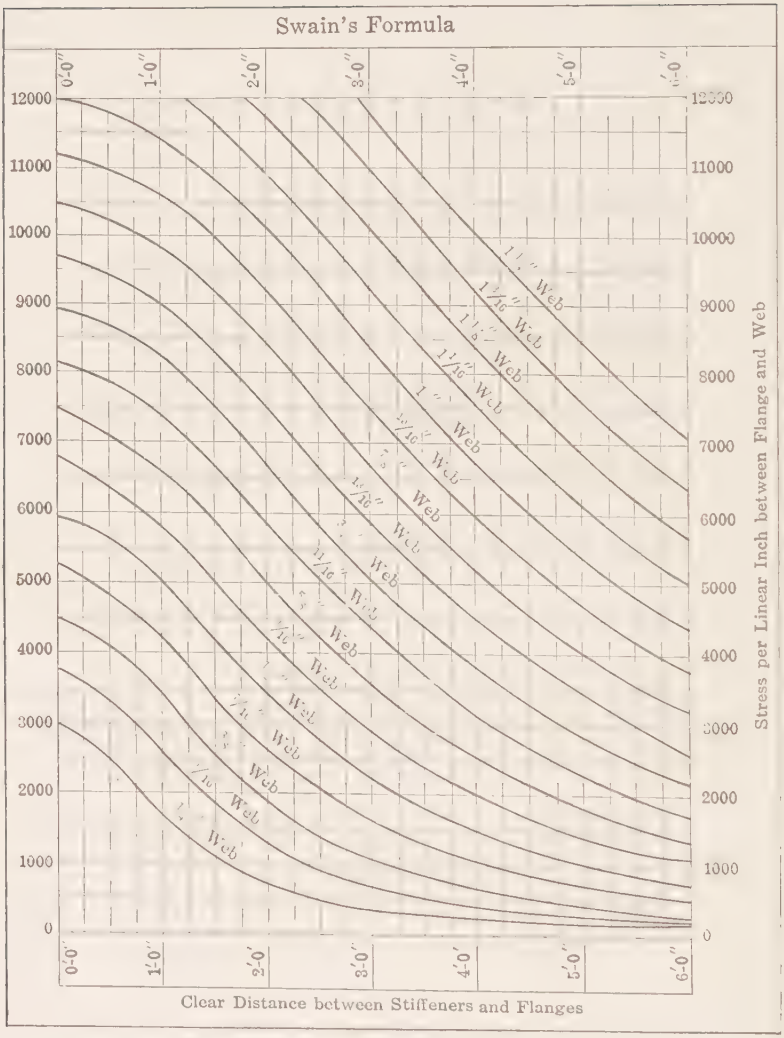

Swain's Formula
0'-0"
1'-0"
2'-0"
3'-0"
4'-0"
5'-0"
6'-0"
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
1 1/2" Web
1 7/16" Web
1 3/8" Web
1 5/16" Web
1 1/4" Web
1 3/16" Web
1 1/8" Web
1 1/16" Web
1" Web
15/16" Web
7/8" Web
13/16" Web
3/4" Web
11/16" Web
5/8" Web
9/16" Web
1/2" Web
Stress per Linear Inch between Flange and Web
Clear Distance between Stiffeners and Flanges

TABLE 26

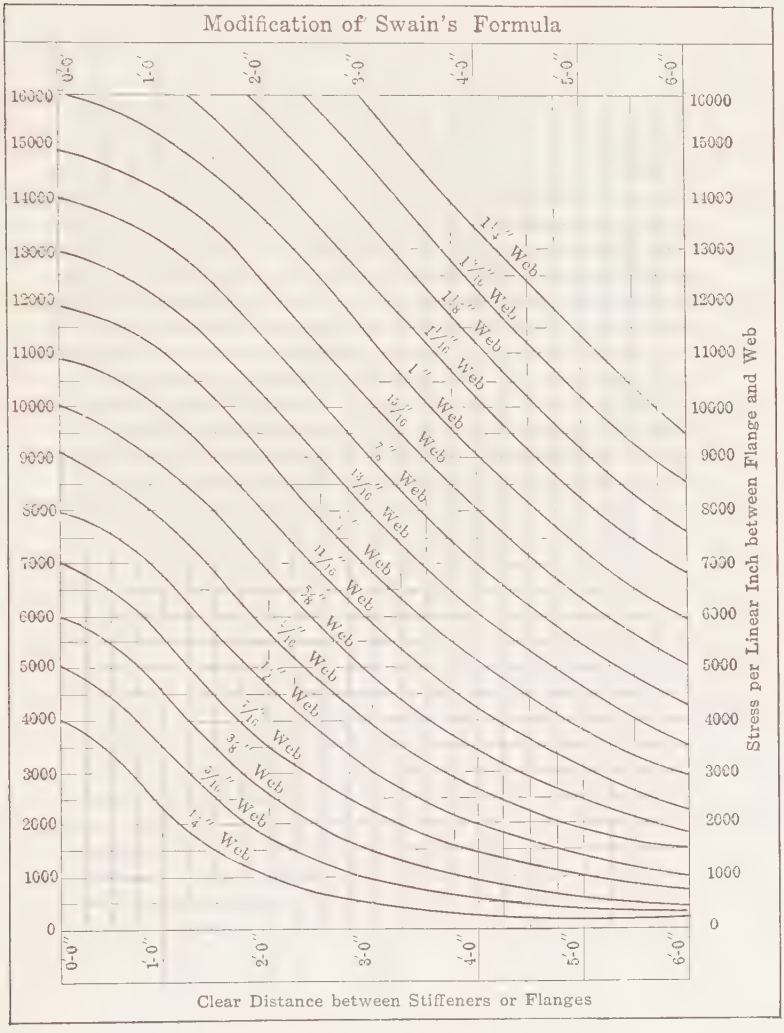

Modification of Swain's Formula
0'-0"
1'-0"
2'-0"
3'-0"
4'-0"
5'-0"
6'-0"
16000
15000
14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
1 1/4" Web
1 3/16" Web
1 1/8" Web
1 1/16" Web
1" Web
15/16" Web
7/8" Web
13/16" Web
3/4" Web
11/16" Web
5/8" Web
9/16" Web
1/2" Web
7/16" Web
3/8" Web
5/16" Web
1/4" Web
Stress per Linear Inch between Flange and Web
Clear Distance between Stiffeners or Flanges

TABLE 27

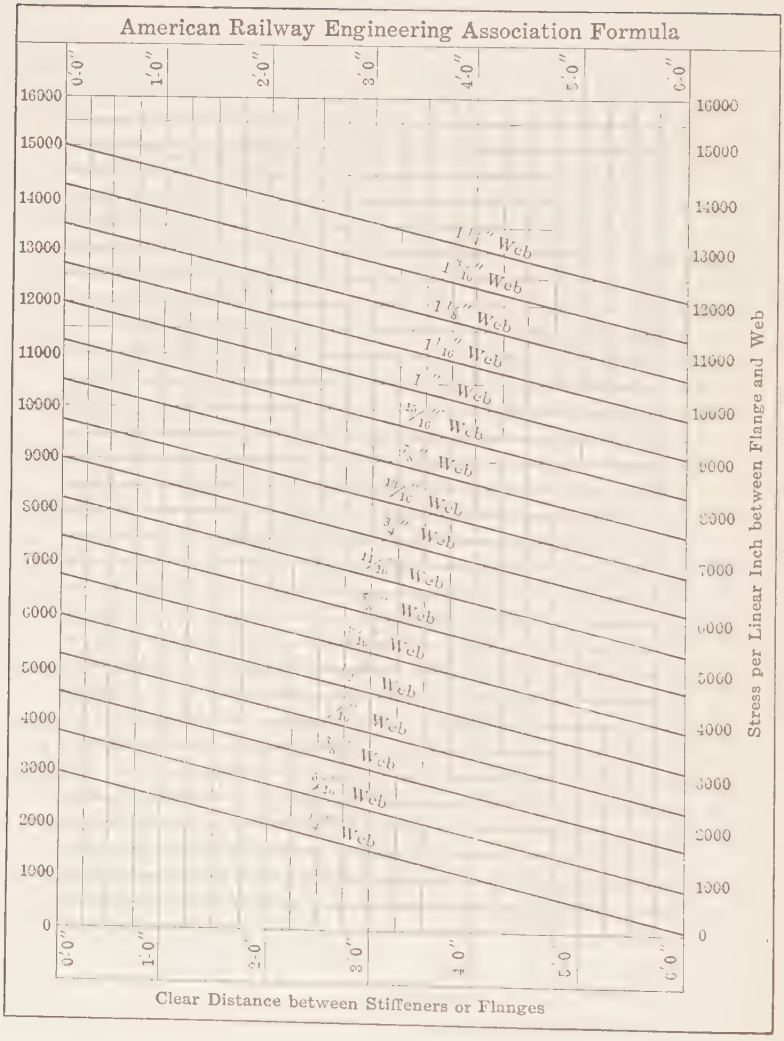

American Railway Engineering Association Formula
0'-0"
1'-0"
2'-0"
3'-0"
4'-0"
5'-0"
6'-0"
16000
15000
14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
1 1/4" Web
1 3/16" Web
1 1/8" Web
1 1/16" Web
1" Web
15/16" Web
7/8" Web
13/16" Web
3/4" Web
11/16" Web
5/8" Web
9/16" Web
1/2" Web
7/16" Web
3/8" Web
5/16" Web
1/4" Web
Stress per Linear Inch between Flange and Web
Clear Distance between Stiffeners or Flanges

TABLE 28

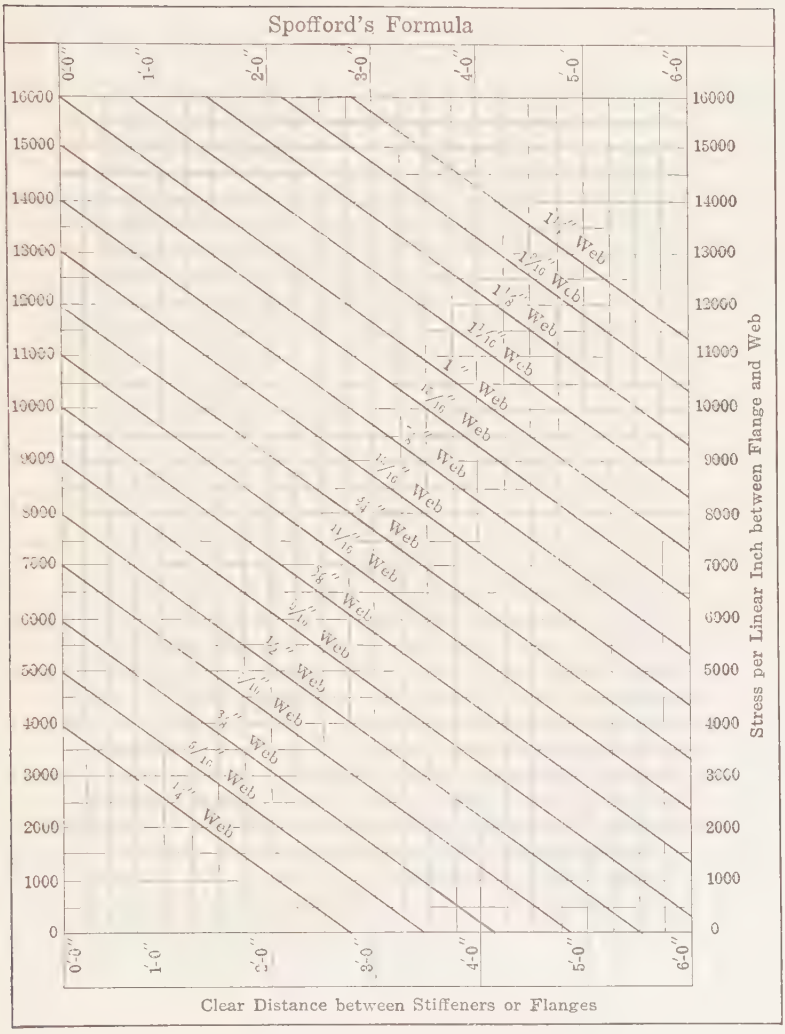

Spofford's Formula
0'-0"
1'-0"
2'-0"
3'-0"
4'-0"
5'-0"
6'-0"
16000
15000
14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
1 1/2" Web
1 7/16" Web
1 3/8" Web
1 5/16" Web
1 1/4" Web
1 3/16" Web
1 1/8" Web
1 1/16" Web
1" Web
15/16" Web
7/8" Web
13/16" Web
3/4" Web
11/16" Web
5/8" Web
9/16" Web
1/2" Web
7/16" Web
3/8" Web
5/16" Web
1/4" Web
Stress per Linear Inch between Flange and Web
Clear Distance between Stiffeners or Flanges

TABLE 29. SHEARING VALUE OF RIVETS.

Diameter of Rivet Inches	Area in sq. ins.	Unit stress = 10000		Unit stress = 11000		Unit stress = 12000		Unit stress = 13500		Unit stress = 15000	
		Single shear	Double shear	Single shear	Double shear	Single shear	Double shear	Single shear	Double shear	Single shear	Double shear
$\frac{3}{8}$	.1105	1105	2209	1216	2431	1326	2652	1492	2983	1658	3315
$\frac{7}{16}$	.1964	1964	3927	2160	4321	2357	4713	2651	5302	2946	5892
$\frac{1}{2}$	.3068	3068	6136	3375	6750	3682	7363	4142	8284	4602	9204
$\frac{9}{16}$	.4418	4418	8836	4860	9720	5302	10604	5964	11928	6627	13254
$\frac{7}{8}$	.6013	6013	12026	6615	13230	7228	14456	8131	16262	9034	18069
1	.7854	7854	15708	8639	17279	9425	18850	10602	21204	11781	23562
$1\frac{1}{8}$	.9940	9940	19880	10934	21868	11928	23856	13419	26838	14910	29820

TABLE 30. BEARING VALUES OF ONE $\frac{3}{4}$-INCH RIVET FOR DIFFERENT UNIT STRESSES AND THICKNESSES OF PLATES.

Unit Stress lbs. per sq. in.	Thickness of Plate.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
10000	1875	2344	2813	3281	3750	4219	4688	5156	5625	6094	6563	7031	7500
12000	2250	2813	3375	3938	4500	5063	5625	6188	6750	7313	7875	8438	9000
14000	2625	3281	3938	4594	5250	5906	6563	7219	7875	8532	9188	9844	10500
15000	2813	3516	4219	4922	5625	6328	7031	7734	8438	9141	9844	10547	11250
16000	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000
18000	3375	4219	5062	5906	6750	7594	8438	9281	10125	10970	11812	12656	13500
20000	3750	4688	5625	6563	7500	8438	9375	10313	11250	12188	13125	14063	15000
22000	4125	5157	6188	7219	8250	9281	10313	11344	12375	13407	14438	15470	16500
24000	4500	5626	6750	7875	9000	10125	11250	12376	13500	14625	15750	16875	18000
25000	4688	5860	7032	8204	9375	10547	11720	12890	14060	15238	16408	17580	18750
26000	4875	6094	7313	8532	9750	10968	12188	13405	14625	15845	17060	18280	19500
28000	5250	6563	7875	9188	10500	11812	13125	14440	15750	17063	18375	19690	21000
30000	5625	7032	8438	9844	11250	12656	14062	15468	16876	18282	19688	21094	22500

TABLE 31. BEARING VALUES OF ONE $\frac{7}{8}$-INCH RIVET FOR DIFFERENT UNIT STRESSES AND THICKNESSES OF PLATES.

Unit Stress lbs. per sq. in.	Thickness of Plate.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
10000	2188	2735	3281	3828	4375	4922	5469	6015	6563	7110	7656	8203	8750
12000	2625	3281	3938	4594	5250	5906	6563	7219	7875	8531	9188	9844	10500
14000	3063	3828	4594	5360	6125	6890	7657	8422	9188	9953	10720	11484	12250
15000	3281	4102	4922	5742	6563	7383	8203	9023	9844	10664	11484	12305	13125
16000	3500	4375	5250	6125	7000	7875	8750	9625	10500	11375	12250	13125	14000
18000	3938	4922	5906	6890	7875	8860	9844	10830	11812	12796	13780	14765	15750
20000	4375	5469	6563	7656	8750	9844	10938	12031	13125	14219	15313	16406	17500
22000	4813	6016	7219	8421	9625	10828	12032	13235	14438	15640	16842	18048	19250
24000	5250	6563	7875	9188	10500	11812	13125	14438	15750	17062	18375	19688	21000
25000	5469	6836	8204	9570	10935	12302	13672	15040	16408	17775	19140	20510	21870
26000	5688	7110	8526	9952	11375	12796	14220	15640	17060	18484	19908	21330	22750
28000	6125	7656	9188	10720	12250	13780	15313	16842	18375	19906	21440	22970	24500
30000	6562	8204	9844	11484	13125	14766	16406	18046	19688	21328	22968	24610	26250

TABLE 32. BEARING VALUES OF ONE 1-INCH RIVET FOR DIFFERENT UNIT STRESSES AND THICKNESSES OF PLATES.

Unit Stresses lbs. per sq. in.	Thickness of Plate.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
10000	2500	3125	3750	4375	5000	5625	6250	6875	7500	8125	8750	9375	10000
12000	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000
14000	3500	4375	5250	6125	7000	7875	8750	9625	10500	11375	12250	13125	14000
15000	3750	4688	5625	6563	7500	8438	9375	10313	11250	12188	13125	14063	15000
16000	4000	5000	6000	7000	8000	9000	10000	11000	12000	13000	14000	15000	16000
18000	4500	5625	6750	7875	9000	10125	11250	12375	13500	14625	15750	16875	18000
20000	5000	6250	7500	8750	10000	11250	12500	13750	15000	16250	17500	18750	20000
22000	5500	6875	8250	9625	11000	12375	13750	15125	16500	17875	19250	20625	22000
24000	6000	7500	9000	10500	12000	13500	15000	16500	18000	19500	21000	22500	24000
25000	6250	7813	9375	10938	12500	14063	15625	17188	18750	20313	21875	23438	25000
26000	6500	8125	9750	11375	13000	14625	16250	17875	19500	21125	22750	24375	26000
28000	7000	8750	10500	12250	14000	15750	17500	19250	21000	22750	24500	26250	28000
30000	7500	9375	11250	13125	15000	16875	18750	20625	22500	24375	26250	28125	30000

TABLE 33. ACTUAL SIZE OF LEGS OF ANGLES.

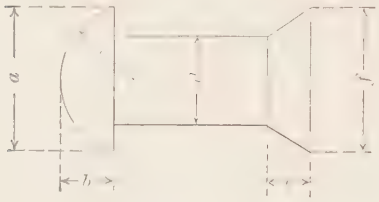
Size.	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
8 x 8							8	$8\frac{1}{16}$	$8\frac{1}{8}$	$8\frac{3}{16}$	$8\frac{1}{4}$	$8\frac{5}{16}$	$8\frac{3}{8}$	$8\frac{7}{16}$	$8\frac{1}{2}$
6 x 6					6	$6\frac{1}{16}$	$6\frac{1}{8}$	$6\frac{3}{16}$	$6\frac{1}{4}$	$6\frac{5}{16}$	$6\frac{3}{8}$	$6\frac{7}{16}$	$6\frac{1}{2}$	$6\frac{9}{16}$	$6\frac{5}{8}$
5 x 5					5	$5\frac{1}{16}$	$5\frac{1}{8}$	$5\frac{3}{16}$	$5\frac{1}{4}$	$5\frac{5}{16}$	$5\frac{3}{8}$	$5\frac{7}{16}$	$5\frac{1}{2}$	$5\frac{9}{16}$	$5\frac{7}{16}$
4 x 4				4	$4\frac{1}{16}$	$4\frac{1}{8}$	$4\frac{3}{16}$	$4\frac{1}{8}$	$4\frac{3}{16}$	$4\frac{1}{4}$	$4\frac{5}{16}$				
$3\frac{1}{2}$ x $3\frac{1}{2}$				$3\frac{1}{2}$	$3\frac{1}{16}$	$3\frac{1}{8}$	$3\frac{9}{16}$	$3\frac{5}{8}$	$3\frac{11}{16}$						
3 x 3			3	$3\frac{1}{16}$	$3\frac{1}{8}$	$3\frac{3}{16}$	$3\frac{1}{4}$	$3\frac{5}{16}$	$3\frac{3}{8}$						
$2\frac{3}{4}$ x $2\frac{3}{4}$			$2\frac{3}{4}$	$2\frac{13}{16}$	$2\frac{7}{8}$	$2\frac{15}{16}$	3								
$2\frac{1}{2}$ x $2\frac{1}{2}$		$2\frac{1}{2}$	$2\frac{9}{16}$	$2\frac{5}{8}$	$2\frac{11}{16}$	$2\frac{3}{4}$	$2\frac{13}{16}$								
$2\frac{1}{4}$ x $2\frac{1}{4}$		$2\frac{1}{4}$	$2\frac{5}{16}$	$2\frac{3}{8}$	$2\frac{7}{16}$										
2 x 2		2	$2\frac{1}{16}$	$2\frac{1}{8}$	$2\frac{3}{16}$										
1 $\frac{3}{4}$ x 1 $\frac{3}{4}$		$1\frac{3}{4}$	$1\frac{7}{8}$	$1\frac{15}{16}$	$1\frac{7}{8}$										
$1\frac{1}{2}$ x $1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{9}{16}$	$1\frac{5}{8}$	$1\frac{11}{16}$	$1\frac{3}{4}$										
$1\frac{1}{4}$ x $1\frac{1}{4}$	$1\frac{1}{4}$	$1\frac{5}{16}$	$1\frac{3}{8}$												
1 x 1	1	$1\frac{1}{16}$	$1\frac{1}{8}$												

OVERRUNS.

AMOUNT BY WHICH EACH LEG EXCEEDS NOMINAL SIZE.

Size.	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
8 x 6						0								
7 x $3\frac{1}{2}$						0								
6 x 4				0										
6 x $3\frac{1}{2}$				0										
5 x 4				0										
5 x $3\frac{1}{2}$			0											
5 x 3			0											
4 x $3\frac{1}{2}$			0											
4 x 3			0											
$3\frac{1}{2}$ x 3			0											
$3\frac{1}{2}$ x $2\frac{1}{2}$		0												
3 x $2\frac{1}{2}$		0												
$2\frac{1}{2}$ x 2	0													
2 x $1\frac{1}{2}$	0													
2 x $1\frac{1}{4}$	0													

TABLE 34. PROPORTIONS OF RIVETS.



Diameter d.	Full Head.			Countersunk.	
	a.	b.	c.	e.	f.
1	1 9/16	39/64	13/16	1/2	1 5/16
7/8	1 3/8	17/32	3/4	7/16	1 1/8
3/4	1 7/32	29/64	5/8	3/8	1
5/8	1 1/32	3/8	17/32	11/32	7/8
9/16	31/32	5/16	7/16	9/32	3/4
1/2	27/32	1/4	11/32	7/32	11/16
7/16	13/16	13/64	5/16	3/16	5/8
3/8	21/32	1/8	9/32	5/32	9/16

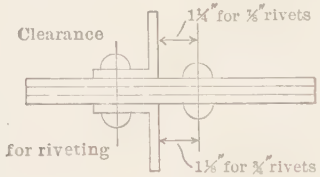
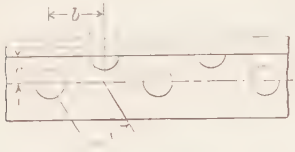


TABLE 35. DISTANCE C TO C OF STAGGERED RIVETS.

Values below or to right of upper zigzag lines are large enough for ¾″ rivets.
Values below or to right of lower zigzag lines are large enough for ⅞″ rivets.

"b" \ "a"	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½
1⅛	1 7/16	1½	1 9/16	1 11/16	1¾	17/8	2	2 1/16	2 3/16	2 5/16	2⅜	2½	2⅝	2¾
1¼	1 9/16	1⅝	1 11/16	1¾	1 13/16	2 1/16	2 1/8	2 3/16	2 5/16	2⅜	2 7/16	2 9/16	2 11/16	2 13/16
1⅜	1⅝	1 11/16	1¾	1 13/16	2	2 1/16	2 3/16	2 5/16	2 7/16	2½	2 9/16	2 11/16	2 13/16	2 15/16
1½	1¾	1 13/16	1 15/16	2	2 1/16	2 1/8	2 5/16	2 7/16	2½	2 9/16	2 11/16	2 13/16	2 15/16	3 1/16
1⅝	1 15/16	2	2 1/16	2 1/8	2 3/16	2 5/16	2 7/16	2½	2 9/16	2 11/16	2 13/16	2 15/16	3 1/16	3 3/16
1¾	2 1/16	2 1/8	2 3/16	2 5/16	2 7/16	2½	2 9/16	2 11/16	2 13/16	2 15/16	3	3 1/16	3 3/16	3 5/16
2	2 5/16	2 3/8	2 7/16	2½	2 9/16	2 11/16	2 13/16	2 15/16	3	3 1/16	3 1/8	3 3/16	3 5/16	3 7/16
2⅛	2 7/16	2½	2 9/16	2 11/16	2 13/16	2 15/16	3	3 1/16	3 1/8	3 3/16	3 1/4	3 5/16	3 7/16	3 9/16
2¼	2½	2 9/16	2 11/16	2 13/16	2 15/16	3	3 1/16	3 1/8	3 3/16	3 1/4	3 5/16	3 7/16	3 9/16	3 11/16
2⅜	2 9/16	2 11/16	2 13/16	2 15/16	3	3 1/16	3 1/8	3 3/16	3 1/4	3 5/16	3 7/16	3 9/16	3 11/16	3 13/16
2½	2⅝	2 11/16	2¾	2 15/16	3	3 1/16	3 1/8	3 3/16	3 1/4	3 5/16	3 7/16	3 9/16	3 11/16	3 13/16

TABLE 36. LEAST STAGGER FOR RIVETS.

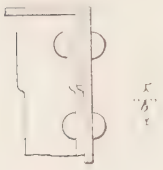


"c" in Inches.	"b" in Inches. For $\frac{3}{4}$" Rivet "a" = $1\frac{1}{8}$"	"b" in Inches. For $\frac{7}{8}$" Rivet "a" = $1\frac{1}{4}$"
$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$
$1\frac{3}{16}$	$1\frac{3}{16}$	$1\frac{7}{16}$
$1\frac{1}{4}$	$1\frac{1}{8}$	$1\frac{3}{8}$
$1\frac{5}{16}$	$1\frac{1}{16}$	$1\frac{5}{16}$
$1\frac{3}{8}$	$\frac{15}{16}$	$1\frac{1}{4}$
$1\frac{7}{16}$	$\frac{7}{8}$	$1\frac{3}{16}$
$1\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{8}$
$1\frac{9}{16}$	$\frac{9}{16}$	1
$1\frac{5}{8}$	$\frac{3}{8}$	$\frac{15}{16}$
$1\frac{11}{16}$	0	$\frac{13}{16}$
$1\frac{3}{4}$	. . . . .	$\frac{5}{8}$
$1\frac{13}{16}$	. . . . .	$\frac{7}{16}$
1	. . . . .	0

TABLE 37. GAGES.

Leg.	Gage g	Maximum Rivet.	Gage g_1	Gage g_2
8	$4\frac{1}{2}$	$\frac{7}{8}$	3	3
7	4	$\frac{7}{8}$	$2\frac{1}{2}$	3
6	$3\frac{1}{2}$	$\frac{7}{8}$	$2\frac{1}{2}$	$2\frac{1}{4}$
5	3	$\frac{7}{8}$	2	$1\frac{3}{4}$
4	$2\frac{1}{2}$	$\frac{7}{8}$	. . . . .	. . . . .
$3\frac{1}{2}$	2	$\frac{7}{8}$	. . . . .	. . . . .
3	$1\frac{3}{4}$	$\frac{7}{8}$	. . . . .	. . . . .
$2\frac{3}{4}$	$1\frac{5}{8}$	$\frac{3}{4}$	. . . . .	. . . . .
$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{4}$	. . . . .	. . . . .
$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{3}{4}$	. . . . .	. . . . .
2	$1\frac{1}{8}$	$\frac{5}{8}$	. . . . .	. . . . .
$1\frac{3}{4}$	1	$\frac{1}{2}$	. . . . .	. . . . .
$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$	. . . . .	. . . . .
$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{3}{8}$	. . . . .	. . . . .
$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{3}{8}$	. . . . .	. . . . .
1	$\frac{5}{8}$	$\frac{1}{4}$	. . . . .	. . . . .

TABLE 38



Rivets in Crimped Angle Distance "b" = $1\frac{1}{2}$ ins. plus twice thickness of flange angles but is never made less than 2 inches.

TABLE 39

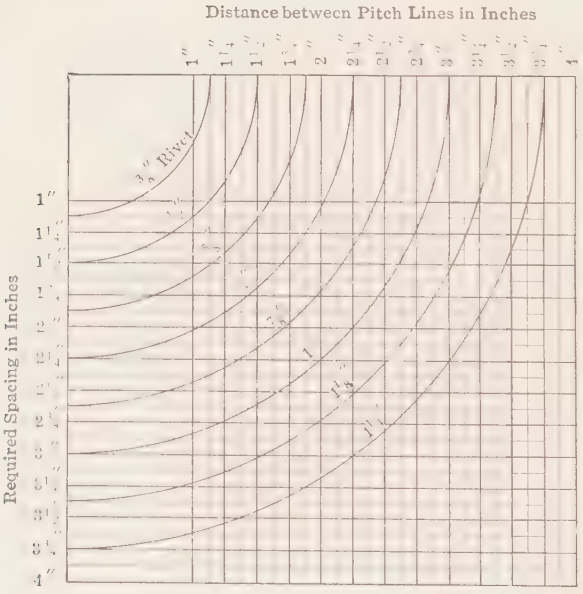


Diagram of spacing for staggered rivets, based on a distance center to center of rivets equal to three diameters.

TABLE 40.—MULTIPLICATION TABLE FOR RIVETSPACING.

Spaces.	Pitch in Inches.														Spaces.
	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½	2⅝	2¾	2⅞	
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1
2	– 2¼	– 2½	– 2¾	– 3	– 3¼	– 3½	– 3¾	– 4	– 4¼	– 4½	– 4¾	– 5	– 5¼	– 5½	– 5¾	2
3	– 3⅜	– 3¾	– 4⅛	– 4½	– 4⅞	– 5¼	– 5⅝	– 6	– 6⅜	– 6¾	– 7⅛	– 7½	– 7⅞	– 8¼	– 8⅝	3
4	– 4½	– 5	– 5½	– 6	– 6½	– 7	– 7½	– 8	– 8½	– 9	– 9½	–10	–10½	–11	–11½	4
5	– 5⅝	– 6¼	– 6⅞	– 7½	– 8⅛	– 8¾	– 9⅜	–10	–10⅝	–11¼	–11⅞	1– 0½	1– 1⅛	1– 1¾	1– 2⅜	5
6	– 6¾	– 7½	– 8¼	– 9	– 9¾	–10½	–11¼	1– 0	1– 0¾	1– 1½	1–2¼	1– 3	1– 3¾	1– 4½	1– 5¼	6
7	– 7⅞	– 8¾	– 9⅝	–10½	–11⅜	1– 0¼	1– 1⅛	1– 2	1– 2⅞	1– 3¾	1– 4⅝	1– 5½	1– 6⅜	1– 7¼	1– 8⅛	7
8	– 9	–10	–11	1– 0	1– 1	1– 2	1– 2	1– 4	1– 5	1– 6	1– 7	1– 8	1– 9	1–10	1–11	8
9	–10⅛	–11¼	1– 0⅜	1– 1½	1– 2⅝	1– 3¾	1– 4⅞	1– 6	1– 7⅛	1– 8¼	1–9⅜	1–10½	1–11⅝	2– 0¾	2– 1⅞	9
10	–11¼	1– 0½	1– 1¾	1– 3	1– 4¼	1– 5½	1– 6¾	1– 8	1– 9¼	1–10½	1–11¾	2– 1	2– 2¼	2– 3½	2– 4¾	10
11	1– 0⅜	1– 1¾	1– 3⅛	1– 4½	1– 5⅞	1– 7¼	1– 8⅝	1–10	1–11⅜	2– 0¾	2– 2⅛	2– 3½	2– 4⅞	2– 6¼	2– 7⅝	11
12	1– 1½	1– 3	1– 4½	1– 6	1– 7½	1– 9	1–10½	2– 0	2– 1½	2– 3	2– 4½	2– 6	2– 7½	2– 9	2–10½	12
13	1– 2⅝	1– 4¼	1– 5⅞	1– 7½	1– 9⅛	1–10¾	2– 0⅝	2– 2	2– 3⅜	2– 5¼	2– 6⅞	2– 8½	2–10⅛	2–11¾	3– 1⅜	13
14	1– 3¾	1– 5½	1– 7¼	1– 9	1–10¾	2– 0½	2– 2¼	2– 4	2– 5¾	2– 7½	2– 9¼	2–11	3– 0¾	3– 2½	3– 4¼	14
15	1– 4⅞	1– 6¼	1– 8⅝	1–10½	2– 0⅜	2– 2¼	2– 4⅛	2– 6	2– 7⅞	2– 9¾	2–11⅝	3– 1½	3– 3⅜	3– 5¼	3– 7⅛	15
16	1– 6	1– 8	1–10	2– 0	2– 2	2– 4	2– 6	2– 8	2–10	3– 0	3– 2	3– 4	3– 6	3– 8	3–10	16
17	1– 7⅛	1– 9¼	1–11⅜	2– 1½	2– 3⅝	2– 5¾	2– 7⅞	2–10	3– 0⅛	3– 2¼	3– 4⅜	3– 6½	3– 8⅝	3–10¾	4– 0⅞	17
18	1– 8¼	1–10½	2– 0¾	2– 3	2– 5¼	2– 7½	2– 9¾	3– 0	3– 2¼	3– 4½	3– 6¾	3– 9	3–11¼	4– 1½	4– 3¾	18
19	1– 9⅜	1–11¾	2– 2⅛	2– 4½	2– 6⅞	2– 9¼	2–11⅝	3– 2	3– 4⅜	3– 6¾	3– 9⅛	3–11½	4– 1⅞	4– 4¼	4– 6⅝	19
20	1–10½	2– 1	2– 3½	2– 6	2– 8½	2–11	3– 1½	3– 4	3– 6½	3– 9	3–11½	4– 2	4– 4½	4– 7	4– 9½	20
21	1–11⅝	2– 2¼	2– 4⅞	2– 7½	2–10⅛	3– 0¾	3– 3⅝	3– 6	3– 8⅝	3–11¼	4– 1⅞	4– 4½	4– 7⅛	4– 9¾	5– 0⅜	21
22	2– 0¾	2– 3½	2– 6¼	2– 9	2–11¾	3– 2½	3– 5¼	3– 8	3–10¾	4– 1½	4– 4¼	4– 7	4– 9¾	5– 0½	5– 3¼	22
23	2– 1⅞	2– 4¾	2– 7⅝	2–10½	3– 1⅜	3– 4¼	3– 7⅛	3–10	4– 0⅞	4– 3¾	4– 6⅝	4– 9½	5– 0⅜	5– 3¼	5– 6⅛	23
24	2– 3	2– 6	2– 9	3– 0	3– 3	3– 6	3– 9	4– 0	4– 3	4– 6	4– 9	5– 0	5– 3	5– 6	5– 9	24
25	2– 4⅛	2– 7¼	2–10⅜	3– 1½	3– 4⅝	3– 7¾	3–10⅞	4– 2	4– 5⅛	4– 8¼	4–11⅜	5– 2½	5– 5⅝	5– 8¾	5–11⅞	25
26	2– 5¼	2– 8½	2–11¾	3– 3	3– 6¼	3– 9½	4– 0¾	4– 4	4– 7¼	4–10½	5– 1¾	5– 5	5– 8¼	5–11½	6– 2¾	26
27	2– 6⅜	2– 9¾	3– 1⅛	3– 4½	3– 7⅞	3–11¼	4– 2⅝	4– 6	4– 9⅜	5– 0¾	5– 4⅛	5– 7½	5–10⅞	6– 2¼	6– 5⅝	27
28	2– 7½	2–11	3– 2½	3– 6	3– 9½	4– 1	4– 4½	4– 8	4–11½	5– 3	5– 6½	5–10	6– 1½	6– 5	6– 8½	28
29	2– 8⅝	3– 0¼	3– 3⅞	3– 7½	3–11⅛	4– 2¾	4– 6⅜	4–10	5– 1⅝	5– 5¼	5– 8⅞	6– 0½	6– 4⅛	6– 7¾	6–11⅜	29
30	2– 9¾	3– 1½	3– 5¼	3– 9	4– 0¾	4– 4½	4– 8¼	5– 0	5– 3¾	5– 7½	5–11¼	6– 3	6– 6¾	6–10½	7– 2¼	30
Spaces.	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½	2⅝	2¾	2⅞	Spaces.
	Pitch in Inches.															

TABLE 40.—MULTIPLICATION TABLE FOR RIVETSPACING—*Continued.*

Spaces.	Pitch in Inches.															Spaces.
	3	3⅛	3¼	3⅜	3½	3¾	4	4⅛	4¼	4⅜	4½	4¾	5	5⅛	5¼	5½	5¾	6	
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1
2	– 6	– 6¼	– 6½	– 6¾	– 7	– 7½	– 8	– 8½	– 9	– 9½	–10	–10½	–11	–11½	1–0	2
3	– 9	– 9⅜	– 9¾	–10⅛	–10½	–11¼	1–0	1– 0¾	1– 1½	1– 2¼	1– 3	1– 3¾	1– 4½	1– 5¼	1–6	3
4	1–0	1– 0½	1– 1	1– 1½	1– 2	1– 3	1–4	1– 5	1– 6	1– 7	1– 8	1– 9	1–10	1–11	2–0	4
5	1 3	1 3⅜	1 4¼	1 4⅞	1 5½	1 6¼	1 8	1 9½	1–10½	1 11¼	2 1	2 2¼	2 3½	2– 4½	2 6	5
6	1–6	1– 6¾	1– 7½	1– 8¼	1– 9	1–10½	2 0	2– 1½	2– 3	2– 4½	2– 6	2– 7½	2– 9	2–10½	3–0	6
7	1–9	1– 9⅜	1–10¾	1–11⅝	2– 0½	2– 2¼	2–4	2– 5¾	2– 7½	2– 9¼	2–11	3– 0¾	3– 2½	3– 4¼	3–6	7
8	2–0	2– 1	2– 2	2– 3	2– 4	2– 6	2–8	2–10	3– 0	3– 2	3– 4	3– 6	3– 8	3–10	4–0	8
9	2–3	2– 4⅛	2– 5¼	2– 6⅜	2– 7½	2– 9¾	3–0	3– 2¼	3– 4½	3– 6¾	3– 9	3–11¼	4– 1½	4– 3¾	4–6	9
10	2–6	2– 7½	2– 8½	2– 9¾	2–11	3– 1½	3–4	3– 6½	3– 9	3–11½	4– 2	4– 4½	4– 7	4– 9½	5–0	10
11	2–9	2–10⅜	2–11¾	3– 1⅛	3– 2½	3– 5¼	3–8	3–10¾	4– 1½	4– 4¼	4– 7	4– 9¾	5– 0½	5– 3¼	5–6	11
12	3–0	3– 1½	3– 3	3– 4½	3– 6	3– 9	4–0	4– 3	4– 6	4– 9	5– 0	5– 3	5– 6	5– 9	6–0	12
13	3–3	3– 4⅝	3– 6¼	3– 7⅞	3– 9½	4– 0¾	4–4	4– 7¼	4–10½	5– 1¾	5– 5	5– 8¼	5–11½	6– 2¾	6–6	13
14	3–6	3– 7¾	3– 9½	3–11¼	4– 1	4– 4½	4–8	4–11½	5 –3	5– 6½	5–10	6– 1½	6– 5	6– 8½	7–0	14
15	3–9	3–10⅞	4– 0¾	4– 2⅝	4– 4½	4– 8¼	5–0	5– 3¾	5– 7½	5–11¼	6– 3	6– 6¾	6–10½	7– 2¼	7–6	15
16	4–0	4– 2	4– 4	4– 6	4– 8	5– 0	5–4	5– 8	6– 0	6– 4	6– 8	7– 0	7– 4	7– 8	8–0	16
17	4–3	4– 5⅛	4– 7¼	4– 9⅜	4–11½	5– 3¾	5–8	6– 0¼	6– 4½	6– 8¾	7– 1	7– 5¼	7– 9½	8– 1¾	8–6	17
18	4–6	4– 8¼	4–10½	5– 0¾	5– 3	5– 7½	6 0	6– 4½	6– 9	7– 1½	7– 6	7–10½	8– 3	8– 7½	9–0	18
19	4–9	4–11⅜	5– 1¾	5– 4⅜	5– 6½	5–11¼	6–4	6– 8¾	7– 1½	7– 6¼	7–11	8– 3¾	8– 8½	9– 1¼	9–6	19
20	5–0	5– 2½	5– 5	5– 7½	5–10	6– 3	6–8	7– 1	7– 6	7–11	8– 4	8– 9	9– 2	9– 7	10–0	20
21	5–3	5– 5⅝	5– 8¼	5–10⅞	6– 1½	6– 6¾	7–0	7– 5¼	7–10½	8– 3¾	8– 9	9– 2¼	9– 7½	10– 0¾	10–6	21
22	5–6	5– 8¾	5–11½	6– 2¼	6– 5	6–10½	7–4	7– 9½	8– 3	8– 8½	9– 2	9– 7½	10– 1	10– 6½	11–0	22
23	5–9	5–11⅞	6– 2¾	6– 5⅝	6– 8½	7– 2¼	7–8	8– 1¾	8– 7½	9– 1¼	9– 7	10–0¾	10– 6½	11– 0¼	11–6	23
24	6–0	6– 3	6– 6	6– 9	7– 0	7– 6	8–0	8– 6	9– 0	9– 6	10– 0	10– 6	11– 0	11– 6	12–0	24
25	6–3	6– 6⅛	6– 9¼	7– 0⅜	7– 3½	7– 9¾	8–4	8–10¼	9– 4½	9–10¾	10– 5	10–11¼	11– 5½	11–11¾	12–6	25
26	6–6	6– 9¼	7– 0½	7– 3⅞	7– 7	8– 1½	8–8	9– 2½	9– 9	10– 3½	10–10	11– 4½	11–11	12– 5½	13–0	26
27	6–9	7– 0⅜	7– 3¾	7– 7⅛	7–10½	8– 5¼	9–0	9– 6¾	10– 1½	10– 8¼	11– 3	11– 9¾	12– 4½	12–11¼	13–6	27
28	7–0	7– 3½	7– 7	7–10½	8– 2	8– 9	9–4	9–11	10– 6	11– 1	11– 8	12– 3	12–10	13– 5	14–0	28
29	7–3	7– 6⅝	7–10¼	8– 1⅞	8– 5½	9– 0¾	9–8	10– 3¼	10–10½	11– 5¾	12– 1	12– 8¼	13– 3½	13–10¾	14–6	29
30	7–6	7– 9¾	8– 1½	8– 5¼	8– 9	9– 4½	10 0	10– 7½	11– 3	11–10½	12– 6	13– 1½	13– 9	14– 4½	15–0	30
Spaces.	3	3⅛	3¼	3⅜	3½	3¾	4	4⅛	4¼	4⅜	4½	4¾	5	5⅛	5¼	5½	5¾	6	Spaces.
	Pitch in Inches.																		

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